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ORDINARY MEETING.

4 April, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

HAL BOYDLE.
JOHN ALFRED CHIPPINDALE.
FRITZ GEORGE DICKINSON, M.C., B.Eng. (*Liverpool*).
HAROLD JOHN BOYER HARDING, B.Sc. (Eng.) (*Lond.*).
CHARLES MELVILLE JENNINGS.
CHRISTOPHER MARRIAGE MARSH, B.Sc. (Eng.) (*Lond.*).
KENNETH GEORGE HOPE MONTGOMERY-SMITH, B.Sc. (Eng.) (*Lond.*).
KEITH RAE SHAND.
ROBERT JOHN SIDDALL.
LESLIE GILBERT TAFFS, B.Sc. (Eng.) (*Lond.*).
STANLEY VAUGHAN, B.Sc. (Eng.) (*Lond.*).
ALEXANDER PROVAN WEIR.

And had admitted as

Students.

TERENCE VICTOR ALLDRITT.
STEWART ARMSTRONG.
RICHARD GED MUIR BATHGATE, B.Sc. (*Edin.*).
ROBERT CRAIG BENNIE.
JOHN ALEXANDER BETTESWORTH.
JOHN LEES BUCKLEY.
JAMES ALLAN BUDGETT.
STANDISH WILLIAM JOHN BUTLER, B.A. (*Cantab.*).
RODERICK CAMERON.
DONALD MCKENZIE CHAPMAN.
EDMOND BENEDICT COUGHLAN, B.E. (*National*).
JOSEPH CURNYN.
RONALD HENRY CUTTS.
NORMAN WILLIAM DENT.
WILLIAM JAMES DEVLIN.
HARRY ELDERS.
BAGHDASAR BEDROS ERDEKIAN.
JAMES REYNOLDS FINK, B.A. (*Cantab.*).
DENYS HAROLD FLETCHER.
WILLIAM STEWART FORBES.
JOHN ERNEST FOYLE.
NEIL GRAHAM.
MEDY DANESH HAERI, B.Eng. (*Liverpool*).
ALAN HARDIE.
DONALD HUTCHISON.
JOHN WILLIAM GRAHAM KERR.

Ronald Henry Leadbeater.
Kenneth Henry Lee.
Karalapillai Loganathan.
Albert Frank Lucarotti.
William Harold Lucas.
Harry Whillis McAughtry.
Archibald William McMurdo, B.Sc. (*Glas.*).
John Norman Merriweather.
James Gordon Moir.
John Alan Fisk Moore.
Thomas Roger Morton, B.Sc. (*South Africa*).
Robert Powell Mumford, B.A.I. (*Dubl.*).
Malcolm Alexander Murchison.
John Donald Bradley Naylor.
Donald Charles Nicholson.
Cyril Herbert Pearsall.
John Mortimer Petty.
William Edward Leon Rennie.
Hugh Charles Gordon Richards.
Francis John Ruoff.
Victor Stanley Sly, B.Sc. (*Birmingham*).
Noel William Smithson.
Robert Walter Snelling, B.A. (*Cantab.*).
Donald Edward Spencer.
Desmond Charles Stephens.
John David Stewart.
Edward Gordon Taylor.
Raymond Taylor.
Huibert van Oosterom.
Ronald Nixon Watson.
Frank Wells.
William Noel Winslade.
Charles James Woodrow.

The Scrutineers reported that the following had been duly elected as

Members.

George James Jackson.
Coningsby Wilde Phillips.
Stanley Livingston Smith, D.Sc. (Eng.) (*Lond.*).

Associate Members.

William Harold Atkinson, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Ronald Bailey, B.Sc. (*Birmingham*), Stud. Inst. C.E.
John Edwin Barnes, Stud. Inst. C.E.
James Philip Barron, Stud. Inst. C.E.
John Stanford Bates, Stud. Inst. C.E.
Oscar Geoffrey Duncan, Stud.Inst.C.E.
Thomas Walker Elliott, B.Sc. (*Glas.*), Stud. Inst. C.E.
Angus Ian Alasdair McCallum, B.Sc. (*Edin.*), Stud. Inst. C.E.
Daniel John Hyslop Morrison, B.Sc. (*St. Andrews*), Stud. Inst. C.E.
Peter Grey Mott, B.A. (*Oxon.*), Stud. Inst. C.E.
John McMillan Murray, B.Eng. (*Sheffield*), Stud. Inst. C.E.
Robert Hume Ogden, M.Sc. Tech. (*Manchester*), Stud. Inst. C.E.
Brian Pennington, Stud. Inst. C.E.
Jack Cobden Purser, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Alexander Frederick Reynard, B.A. (*Cantab.*), Stud. Inst. C.E.
Andrew Wilson, B.Sc. (*Aberdeen*), Stud. Inst. C.E.

The following Papers were submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5100.

"Investigation of the Outer Approach-Channels to the Port of Rangoon by means of a Tidal Model."†

By Oscar Elsden, M.Sc., Assoc. M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

For some years past, certain areas of the approach channels to the Port of Rangoon appear to have been deteriorating to an abnormal extent, a process which would threaten the prosperity of the Port if continued unrelieved and unchecked.

In 1929 the Port Commissioners invited Sir Alexander Gibb and Partners to state their views regarding a programme for future study and observation of the approach channels across the Outer bar. Field surveys and investigations were started, and in 1931 it was decided to complete these by means of tidal-model experiments.

Conditions in the Irrawaddy Delta and Rangoon Estuary.

The present Paper describes the tidal-model experiments only, but, to provide a clear picture of the whole problem, some introductory information about the conditions of the Rangoon river is given.

Rangoon, the capital of Burma, is situated upon the left bank of the Rangoon river, 25 miles from its mouth at Elephant Point (Fig. 1, Plate 1). The river provides deep-water connexion with the sea and with the thousand or so miles of navigable waterways in the Irrawaddy delta. Railways and roads connect Rangoon with districts not served by natural waterways; the city is the commercial centre of Burma, and now handles 90 per cent. of its overseas trade.

The port is administered by a Port Commission, whose jurisdiction

† Correspondence on this Paper can be accepted until the 15th August, 1939.—Sec. Inst. C.E.

extends over the river from about 10 miles above Rangoon to about 10 miles south of Elephant Point, and also covers the entrance to the China Bakir river, another navigable mouth of the Irrawaddy (Fig. 1, Plate 1). The upland waters of the port of Rangoon are provided by the Irrawaddy river and by the Rangoon river.

The Irrawaddy drains an area of about 160,000 square miles. Owing to the monsoon it is subject to wide variations of flow, and to heavy floods in its lower reaches. Its headwaters run through districts of fairly hard geological formation, and above Mandalay the silt-content of the river is comparatively light. The lower tributaries, however, rise in softer areas, causing the silt-contents to become appreciable, and as a result, the Irrawaddy has built up a delta (Fig. 1, Plate 1), which starts near Henzada, 150 miles from its present mouths. The Bassein and Rangoon rivers are its western and eastern extremes, and the coastline between their mouths is about 160 miles long. The area of the delta is 21,000 square miles, of which about one-third is drained by the Rangoon river.

The Rangoon river (in its upper reaches known as the Myitmaka) rises in the Inma lake and flows parallel with the Irrawaddy, often at distance of only a few miles. Lower down it is known as the Hlaing, until, within a few miles of Rangoon, it becomes the Rangoon river. On its left bank are the Pegu Yomas, hills formed of soft clay and sandstone, and the numerous tributaries from these hills bring great quantities of silt into the Myitmaka-Hlaing during flood seasons.

Both the Irrawaddy and the Rangoon rivers have several interconnexions, as the map shows; some are permanent, some seasonal. During the Monsoon season there is a large transfer of water from the Irrawaddy; this transfer is, at its maximum, greater than the whole run-off from the catchment area of the Rangoon river. About 4 miles above Rangoon the Rangoon river is joined by the Panhlaing creek, by which at all seasons it receives water from the Irrawaddy. The Pegu river, draining about 2,000 square miles of the east slopes of the Pegu Yomas, enters just below the city of Rangoon.

It is necessary to emphasize that the upland water discharge down the Rangoon river is derived from these very different sources, and that the conditions affecting the regime of the river vary greatly between the different seasons.

The formation of the Rangoon river seems to be of very recent growth, since during the sixteenth century the outlet of the Myitmaka-Hlaing river was via the lower part of the Pegu, which discharged further east, close to the present mouth of the Sittang. The Rangoon river in its present alignment certainly existed by the middle of the seventeenth century, but only became navigable in the latter part of the eighteenth century.

The discharges of water and of silt down the Myitmaka-Hlaing and Panhlaing rivers were gauged in 1912–15 by the late Mr. E. C. Niven, M. Inst. C.E., then Executive Engineer to the Port, and again in 1930–33

during a hydrometric survey made by the Port Commissioners' technical officers. The figures show that during the dry season the average discharge down the Rangoon river is about 12,000 cusecs; this increases to about 170,000 cusecs during the wet season, while peak floods may bring the total to about 300,000 cusecs.

Typical results of the silt gaugings at the Kemmendine gauging station (near the junction of the Myitmaka-Hlaing with the Panhlaing), are as follows :—

Content of dry silt: grains per cubic foot of water.

		Ebb tide.	Flood tide.
Dry season :	Neap tides	579	672
	Spring tides	1,050	1,052
Monsoon :	Neap tides	256	185
	Spring tides	290	206

The silt-contents are obviously fairly high, and the other mouths of the Irrawaddy also carry much suspended matter.

The Sittang river[1] must also be mentioned, although it is not directly connected with the Rangoon. Lying east of the Irrawaddy delta, it drains a total area of about 13,500 square miles into the head of the Gulf of Martaban. Its discharge, estimated from rain-gauge readings, is about 8,000 cusecs during the dry weather and about 150,000 cusecs during the monsoon, when it carries a very great deal of fine silt. Its mouth is blocked with uncharted shoals, navigation is practically impossible, and there is a bore sometimes reaching a height of 6 feet. As a result, very little information is available about the river. Its importance in the present connexion is due to its effect on the tides near the mouth of the Rangoon river and its production of shoals in the head of the Gulf of Martaban.

The general depth of the Gulf of Martaban is less than 20 fathoms, most of it being less than 10 fathoms deep at low water. The head of the gulf is uncharted, but is known to be shoaling fairly rapidly, and the coastline is advancing at the rate of about 3 miles a century. The tidal wave flows up the gulf in approximately a north-west–south-east line, travelling at about 14 knots, and giving rise to streams which may reach a velocity of 7 knots, and which, in the northern part of the gulf, run parallel to the coastline. The converging coastlines cause a rapid increase in the range of the tidal wave as it passes eastward. The spring-tide range is 18 feet at Elephant Point, and 22 feet at the head of the gulf, and the neap-tide range varies similarly. The tidal wave shows a marked diurnal variation which may lead to a difference of as much as 4 feet between the ranges of successive tides.

[1] J. M. B. Stuart, "The Sittang River and its Vagaries." Inst. C.E. Selected Engineering Paper No. 127. 1932.

Fig. 2, Plate 1, which shows the Rangoon river seaward of the port, is based on the Port Commissioners' survey of 1932. It will be seen that the river meets the sea at Elephant Point, where it has a width of about 3 miles at high water. About 6 miles above this point, the channel is divided into two arms, separated by a shoal known as Middle bank. The easternmost of these channels continues straight out to sea, maintaining a south-easterly direction. The western channel starts roughly parallel to the eastern channel, but at Elephant Point it turns, somewhat abruptly, to the south-west, and runs across a shallower area to deep water. It is this shallower area lying south-west of the mouth of the river that is known as the Outer bar. The sands separating the seaward ends of these channels are known as the Eastern sands.

The eastern channel is straight, and has at present a depth of 24 feet at low-water, but it is narrow, and is subject, when the eastern sands are covered, to a cross tide approaching 6 knots at spring tides. Ships must therefore use the western channel, which has an average ruling depth of about 19 feet at low-water spring tides, sufficient for all ordinary traffic of the port.

Though the general topographical regime of the Rangoon river has remained unaltered for a very long period, there have been, at any rate since 1875, noteworthy changes in individual features. The main changes are :—

(1) The west bank of the river at Elephant Point has been eroded continuously, and there has resulted a marked straightening of the 6 miles of western channel immediately above Elephant Point, and a widening of the river mouth from about 2 miles in 1875 to about 3 miles in 1932.

(2) This erosion has been accompanied by a steady widening in the Middle bank, so that the cross section of the western channel has remained very nearly unchanged.

(3) The Middle bank has extended lengthwise in both directions, and is now practically connected with the eastern sands.

(4) Elephant Point, and the high-water mark along the China Bakir flats, have advanced seawards by about a mile.

(5) The eastern channel has steadily narrowed and has swung slightly to the east.

In spite of these changes, the access to the port remained without serious threat until about 1910, when the Outer bar first began to deteriorate. By 1931, when the Commissioners decided to construct a model, the ruling depth over the Outer bar ranged from 12 feet to 15 feet at low water (over a width of between 5 and 7 miles). Though there is here a spring tide range of 18 feet, any further deterioration would greatly handicap shipping.

Design, Construction, and Adjustment of the Model.

When model-investigations were proposed, there were in operation in Great Britain tidal models of the Severn [1] and Mersey estuaries, constructed by Professor A. H. Gibson, D.Sc., M. Inst. C.E., who had also constructed a model of the Humber. Further afield, Mr. John McClure, B.Sc. (Eng.), M. Inst. C.E., had constructed a model of Bombay harbour in which he successfully investigated problems relating to dredging, silt distribution, and the travel of sewage [2]; and Mr. W. C. Ash, B.Sc. (Eng.), M. Inst. C.E., had adopted model experiments to study construction and dredging problems in the port of Vizagapatam[3]. Similar experiments with models had been made on the Continent, and in the United States of America.

Although the design of the Rangoon model owed a great deal to these experimenters, it was necessary to make certain material departures from previous practice. The channel which it was intended to study lay not in a river but in an open tideway, and was subject to the possible influence, not of one river, but of a number, forming a complex delta-system; further, conditions in these rivers were subject to important seasonal fluctuations. The foregoing description of the delta-system in which the Port is situated shows the complexity of the problem of analyzing the appearance and growth of the Outer bar deposits, which might possibly be ascribed to one or more of the following causes:—

(1) Silt brought down by the Rangoon river.

(2) Silt falling into this river as a result of continual erosion of its western bank near Elephant Point.

(3) Silt brought down by the other mouths of the Irrawaddy and washed eastwards by tidal streams.

(4) Silt brought down by the Sittang, possibly increased in amount or altered in distribution by the sudden changes of 1906, when the Alok cut was formed.

(5) Changes in the tidal streams due to the widening, straightening, and other changes in the Rangoon river mouth.

(6) Other unknown factors.

The purpose of the tidal model was to find the cause of the growth of the Outer bar, the extent to which it would probably develop if left unchecked, and the most economical means by which a permanent navigable channel could be maintained through it. In order that all the important factors should be represented, and all disturbing effects due to artificial boundaries as far removed from the Outer bar as possible, it was decided

[1] A. H. Gibson, "Construction and Operation of a Tidal Model of the Severn Estuary." H.M. Stationery Office, London, 1933.

[2] J. McClure, "Bombay Harbour Survey and Tidal Model." Minutes of Proceedings Inst. C.E., vol. 232 (1930–31, Part 2), p. 66.

[3] W. C. Ash and O. B. Rattenbury, "Vizagapatam Harbour." Journal Inst. C.E., vol. 2 (1935–36), p. 235. (December 1935.)

that the model should include as large an area around the Outer bar as was practicable; the tidal compartments of the Rangoon, China Bakir, and Sittang rivers; and the Bassein creek and Twante canal connecting the first two of these rivers. Within this area it was intended to reproduce those natural phenomena which might be considered to have any bearing upon the problem. These were:—

(1) The tidal wave in the Gulf of Martaban, including the spring–neap cycle and diurnal variation.
(2) The flows of water and of suspended silt down all rivers within the modelled area, and the littoral drift of silt into that area from the rivers which lie to the westward of it.
(3) The erosion of certain stretches of the coastlines, and the travel of silty material from this source.
(4) The accelerated settlements of silt deposits under the influence of saline sea water.
(5) The seasonal variations in the water and silt discharges, and the effect of the south-west monsoon wind.

The Governing Committee of University College, London, most generously placed a large basement at the Commissioners' disposal, and the engineers were given the fullest opportunity of co-operating with the Staffs of the Engineering, Chemical, and Physical departments of the College. Two rooms were adapted, each about 40 feet by 60 feet; one to accommodate the model, the other being retained as workshop, store-room and office. This accommodation permitted a suitable area to be reproduced to a scale of 9 inches to 1 sea mile, or 1 : 8068. The vertical scale was derived from the empirical formula obtained by Professor Osborne Reynolds, which gives a relationship between the actual tide range in an estuary and the vertical scale exaggeration in its model[1]. From these two linear scales, the time-scale is evolved by use of another empirical formula of Professor Reynolds, which determines the ratio between the actual and model tidal-periods. The remaining scales of horizontal and vertical velocity follow directly from the time-scale and the two scales of length. These principles have been adopted in Great Britain in all tidal models as yet constructed. In the Rangoon model, the various scales were:—

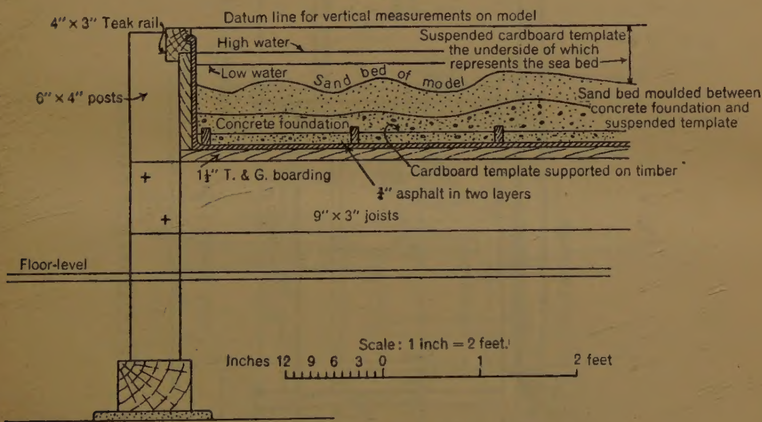
Length	1 : 8060 horizontal and 1 : 192 vertical.
Time:	1 : 583.
Velocity:	1 : 13·85 horizontal and 3·04 : 1 vertical.
Volume:	1 : 12,500 $\times 10^6$.
Discharge:	1 : 21·5 $\times 10^6$.

[1] Osborne Reynolds, "On the Action of Waves and Currents." Scientific Papers, Vol. 2, p. 380; *also* L. F. Vernon-Harcourt, "The Principle of training Rivers through Tidal Estuaries, as illustrated by Investigations into the Methods of Improving the Navigation Channels of the Estuary of the Seine." Proc. Roy. Soc., vol. XLV (1888–89), p. 315.

Fig. 3, Plate 1, is a general plan of the model as constructed. It was moulded in a special wooden tank lined with asphalt, laid on felt and expanded metal. The tank had sides 15 inches high and was of an irregular shape on plan. The floor and sides were supported on a substantial wooden framework, whose supports passed through the existing wooden floor of the laboratory on to solid foundations beneath: every care was taken to avoid settlement or distortion which would affect the results of working the model. *Fig. 4* is a typical section through the model, giving constructional details.

A chamber was formed at the western end of the model tank to accommodate the plungers of the tide generator, and a drain at this end enabled the whole model to be emptied. A second drain was provided

Fig. 4.

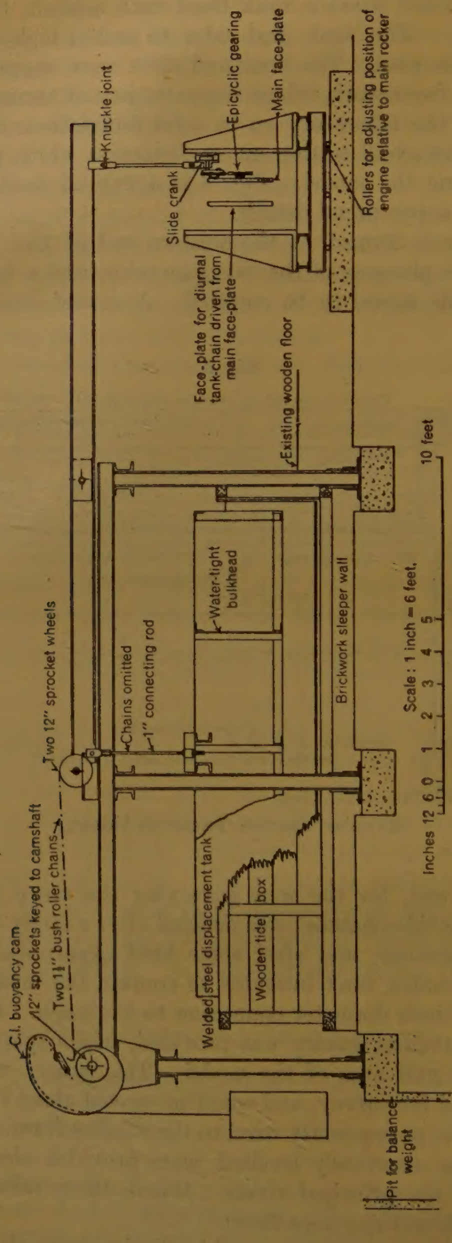


Typical Section Through Model.

at the opposite end, for the weir governing the mean tide-level. The orientation of the tide-chamber was decided after a study of the directions of actual tidal streams, and after some brief experiments with a small replica of the wooden tank intended to contain the model. The model was filled by a 2-inch diameter connexion to the main. For survey, and other work, a travelling gantry was provided, which spanned the shorter dimension of the gulf area of the model. This gantry carried a 19-foot straight-edge used for survey, and could be moved along two rails formed of mild-steel angle, permanently fixed to the wooden framing of the model. Teak datum rails, accurately levelled, were provided along each side of the gulf end of the principal rivers. Where these rails spanned river mouths, moveable sections were fitted.

The model tidal wave was generated by displacement plungers, operated by a mechanism similar to that previously used by Professor Gibson, part

Fig. 5.



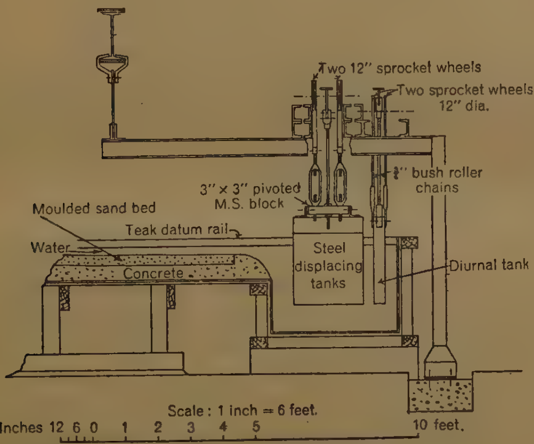
Tide-Generating Machinery.

of the machinery being, in fact, obtained from one of his disused models. The tidal machinery is illustrated in *Figs. 5* and *6*.

Two plungers were used; a main tank to generate the half daily tide, and a diurnal tank, running at half the speed, to give the diurnal component. The sizes of these tanks were computed from the tidal volumes of the areas which were to be included in the model. Both tanks were of mild steel, welded throughout, and coated with bitumastic paint. Each was divided by transverse bulkheads into four equal watertight compartments, to facilitate trimming with iron and water ballast.

Each tank was suspended from, and driven by, an overhead rocking beam directly connected to the tide-engine, which drove the main tank by a rotating face-plate carrying an epicyclic train of gears. Upon the

Fig. 6.



SECTION SHOWING TIDE-GENERATING MACHINERY.

last wheel of this train was carried a crank-pin working in a horizontal slide-crank, which it caused to move up and down in a guide formed in a vertical pillar (*Fig. 7*, p. 12). The slide-crank was connected by a driving rod to the rocking beam for the main tank. The epicyclic gearing was arranged to alter the vertical stroke of the slide-crank, and thus of the tide tank, from its maximum to its minimum in fourteen revolutions of the face-plate, thus causing the tide generated to vary from a spring range to a neap range in the correct number of tides. The rocking beam carrying the diurnal tank was operated by a similar slide-crank and face-plate mechanism, which was driven by a chain drive from the main face-plate. Provision was also made on the diurnal face-plate for an epicyclic train which would introduce periodic changes in the diurnal variation.

The main face-plate was driven by a $2\frac{1}{2}$-horse-power direct-current motor through totally-enclosed double-reduction worm gearing. The main gear-

box, base-plate, and pedestal for the main face-plate, and the pedestal for the diurnal face-plate, were of cast iron, as also were the two face-plates themselves, and the two guide-pillars. The gears were cut from mild steel and the worm drives ran on ball races. The epicyclic gears were also cut from mild steel, and ran on bronze bushes on steel pins screwed into the face-plates. The two slide-cranks were made up from structural angles, machined where necessary, and their die-blocks were of bronze. Grease lubrication was fitted to all points.

The whole machine was mounted on a substantial framework of structural steel, which was carried on rollers, permitting it to be moved

Fig. 7.

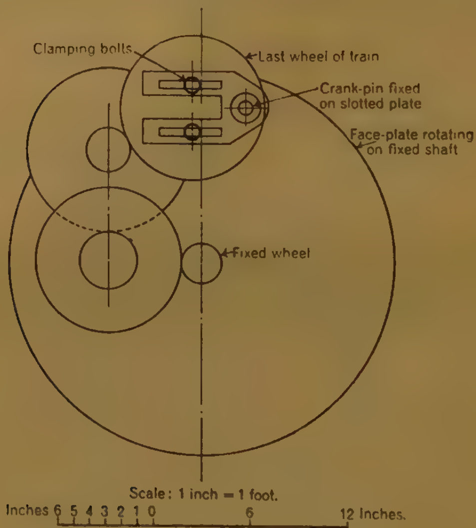


Diagram of Epicyclic Train.

along slide rails set in a concrete bed. It could thus be moved relative to the fulcrum of the main rocker, so that the range of tide could be properly adjusted. Clamp-bolts were provided to keep the engine at this setting. The main crank-pin was mounted in a slotted plate; it was possible to alter the distance of the crank-pin from the centre of the gear-wheel upon which it was mounted, and thus to adjust the spring-tide–neap-tide ratio. A similar adjustment was fitted to the diurnal crank-pin.

To keep the tanks balanced at all positions, two balance-weights were provided, one for each tank. Each weight was hung by a wire rope from a large grooved cast-iron cam, which was driven by roller chains connected to the appropriate tank. As the tank rose out of the water during an ebb tide, for instance, the cam increased the lever arm of the balance-weight to compensate for the loss in buoyancy of the tank, and thus kept a per-

fectly steady load on the main engine. The buoyancy-variation amounted to about 4,500 lb., and, with the 4-to-1 cam-ratio provided, the weight of main tank and ballast had to be about 6,000 lb., the main balance-weight being about 1,400 lb.

The camshafts, rocking beams, and other parts of the overhead gear were supported by a substantial structural steel-frame carried by five stanchions and one roof-hanger. The main tank balancing gear was carried on roller bearings; plain brass bearings were used for the light loads imposed by the diurnal tank. The roller chains connecting the cams to the tanks were lubricated with thin oil supplied by wick-feeds. The grooves in the two cams for the balance-weights were lined with red fibre, to avoid undue wear in the ropes carrying the balance-weights. Bottle-screws, provided between each tank and the roller chains connecting it to its balancing cam, altered the setting of the cam, and enabled the final balance of the tanks to be very delicately adjusted. The main motor had a no-volt and overload trip, and was connected to the mains through two variable resistances, so that its speed could be accurately adjusted and maintained against the varying electrical demands of the neighbouring College laboratories.

In order to control automatically the seasonal variations in the various discharges of silt and fresh water, and in the mean sea-level, a timing gear was provided on the main engine. The device consisted of a reduction gear rotating a shaft once for every seven hundred and four tides, or 1 "model-year." This shaft carried two cams, each of which closed an electric switch for half a revolution of the shaft or for the model equivalent of 6 months, and the two cams were set so that when the one switch was closed the other was open. The first cam closed an electric circuit and caused a number of small motors to bring the model under monsoon conditions for 6 "model-months." At the end of this period the "monsoon circuit" was broken, and the second cam closed the "dry weather circuit", bringing all the model-rivers, etc., to dry-weather positions. The seasonal variations were thus rendered independent of manual control, and the model could be left unattended when running at nights without affecting the experiments.

Six rivers were included in the model. Each had a separate measuring vessel with two compartments and a measuring orifice in the bottom of each compartment, one to supply the average dry-weather flow, the two together to discharge the average monsoon flow down the river. Each river-measuring vessel was fitted with a gauge-glass and scale, the scales being marked after calibration in the Hydraulics laboratory of the College. The discharges of the model-rivers were based on the field gaugings to which reference has been made, or on calculations from rainfall. In the case of the Sittang, only one-half of the calculated discharge was supplied to the model, for it was considered that only half the actual river could be said to flow into the modelled area. All the rivers received their water

from a 250-gallon tank connected to the main by a ball-valve. Two separate pipelines delivered water to each measuring vessel. The first was running continuously, and was adjusted by a screw-valve to deliver the correct dry-weather flow. The second pipeline was similarly adjusted, so that its discharge brought the flow of the model-river up to the mean monsoon rate, and was opened by the timing gear during "monsoon seasons." The proper periods of monsoon and dry-weather flow were thus obtained. As actually arranged, the monsoon pipelines for all six rivers were controlled by one direct-current motor of $\frac{1}{20}$ horse-power.

Silt was discharged into the rivers at rates determined from the field observations. A hopper-and-belt feed carried the silt into the river supply tank, where the silt-laden water was kept agitated by a propeller. The rate of silt fed to the tank was varied between monsoon and dry seasons by means of the timing gear on the main engine, which increased the speed of the belt-feed during the monsoon.

The littoral movement of silt from the more western mouths of the Irrawaddy was reproduced in the model by an additional feed of silty water. The rate at which this "tidal" silt was introduced again followed from field observations made for that purpose by the Port Commissioners' staff, who took extensive series of silt gaugings in the real Gulf of Martaban on flood and ebb tides, and determined the silt-contents at a number of standard stations under all tidal and seasonal conditions. From the difference between flood-tide and ebb-tide silt-contents it was possible to compute the rate at which silt was entering the modelled area from the westward, and thus to decide the amount of silt which should be fed into the model from this source during the course of each model-tide.

This tidal silt was supplied to the model from a tank similar to that used for the supply of the model-rivers. In this tank the silt was stirred up in water carried along a perforated trough spanning the model immediately in front of the main tide-plunger. The spacing of the perforations was varied, so that the greater part of the silt was discharged fairly close inshore, and decreasing amounts further seaward; the field observations showed that there was such a turbidity-gradient. Checks made during the operation of the model at places in the model-gulf corresponding with the actual gauging stations gave silt-distribution results which agreed well with those recorded in the real Gulf.

It was essential that the silt in the model should settle at the proper scale rate, and that the model should allow for the sudden acceleration of settlement which occurs in nature when a stream of dirty river water flows into the sea. Following the method adopted by Professor A. H. Gibson in the river Severn model, a coagulant solution was used for this purpose. "Actual settlement-rates" were determined by making up suspensions of Rangoon silt in Rangoon sea-water in standard proportions, and noting the times necessary for these suspensions to settle. These tests were carried out in Rangoon by the Port Commissioners' engineers.

Similar tests were repeated in the model-laboratory, using similar suspensions of commercial clays in coagulant solutions, and a series of "model settlement-rates" was found. The summary of scales on p. 8 shows that vertical velocity in the model must be about 3 times that in nature; thus the "model settlement-rate" should, correctly, be 3 times the "actual settlement-rate."

In the latter series of tests, two commercial clays were selected, of which uniform supplies could be obtained in suitably finely divided form. Their settlement-rates were tried in coagulants of different forms and concentrations to find the most suitable coagulant for use.

The proper settlement was obtained by introducing potassium alum to the model-sea in sufficient quantity to maintain a density of 1·0025. The alum was dissolved in perforated trays suspended in the tank which supplied the tidal silt to the Gulf.

In order to keep the model-sea at its proper mean level a weir was fitted at the position shown in Fig. 3, Plate 1. The weir was formed of a vertically-sliding brass plate, and had a crest 18 inches long. It was arranged so that the crest of high water just spilled over into the drain provided for the purpose, and could be adjusted so that the dry-weather mean level was properly maintained against the discharge of water down the model-rivers. When these discharges increased during the model-monsoon, the weir was slightly lowered by means of a $\frac{1}{20}$ horse-power direct-current motor; this was controlled by the timing gear on the main engine, and also raised the weir when the model-rivers returned to their dry-weather discharges. The lever arms from which the weir was suspended could be adjusted, and it was found possible in practice to maintain the model sea-levels at values closely corresponding to actual figures.

The Elephant Point tide-gauge was chosen as the measuring point for mean sea-level, which at this point had been established in the relation to the datum for soundings in the Gulf of Martaban. The slight seasonal variation in mean tide-level at this gauge, due to the Rangoon river flow [1], was reproduced in the model.

The south-west monsoon winds cause considerable sea disturbance in the gulf. The appropriate waves were produced in the model by a battery of seven fans, connected to the seasonal control-gear which operated them for the proper periods. The fans were adjusted to create waves travelling in the proper direction, and, on the advice of Professor Gibson, the strength of wind produced by the fans was adjusted until the model-waves had the correct height from crest to trough, a factor considered to be of more importance in this case than length or speed.

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[1] Tide Tables of the Indian Ocean. From Admiralty Tide Tables, 1938, Tidal Predictions. Part 1, Section B.—Foreign Waters (The World, except British Islands and North and West Coasts of Europe). Published by His Majesty's Stationery Office for the Hydrographic Department, Admiralty.

For moulding, and for subsequent survey-work, the datum rail was marked off into a system of permanent section lines; over the Outer bar area these were $\frac{1}{4}$ mile apart, this interval being increased in the less important areas. This system of section-lines was referred to the Shwe Dagon pagoda at Rangoon and to Eastern Grove lighthouse at the Rangoon river mouth, two landmarks whose latitude and longitude were known, and which were represented in the model by permanent steel pegs. The section-lines were plotted on all actual charts.

Much of the model was permanently moulded in concrete, in order to save time in the frequent remouldings to standard conditions. The China Bakir river, Bassein creek, and Twante canal were completely moulded in this manner, as were the more permanent features of the Rangoon river. Space was left in the Rangoon river for the introduction of softer materials with which to reproduce past erosion of its banks and that which might be estimated to occur in the future. The whole bed of the model, as moulded in concrete, was kept below the levels shown on the charts and there was a final layer of sand, 3 inches, or about 50 scale feet, thick. Round the mouth of the Rangoon river the thickness of this sand bed was increased to 100 scale feet, in order to give room for considerable scour which normally occurred, and which might be increased by the construction of hypothetical engineering works. The lands between the various rivers consist mainly of low-lying paddy fields; their level is generally below that of high water, and they are protected from flooding by bunds, though heavy flooding does occur in the monsoon season. These features were reproduced in the model, in case a study of the effects of abnormally high tides, or of flooding, should be required.

The concrete moulding was done by means of cardboard templets permanently fixed to the asphalt floor of the model-tank. The spaces between these templets were packed with coke-breeze with about 2 inches of concrete on top, the surface of the concrete being brought accurately up to level by a rendering of 1 : 4 cement mortar. More intricate parts of the Rangoon river were moulded in paraffin wax.

For the moulding of the overlying layer of sand, cardboard templets were cut to the profiles shown on the natural charts. These were suspended from above the model by a wooden frame (*Fig. 4*, p. 9). The sand spread over the model was made fairly damp and was accurately moulded to these profiles by hand and with the aid of trowels. The amount of sand in the model was about 14 tons, and the complete moulding required about 10 days' continuous work.

The converging coastlines of the Gulf of Martaban cause a marked increase in the range of the tidal wave as it travels eastward; this funnel effect was reproduced in the model by forming the seaward boundary as shown in Fig. 3, Plate 1. The portion of the tank intended to contain the model gulf was made rectangular, and the additional internal boundary was put in after the whole tank had been asphalted; provision was also

made to alter its alignment should it fail to produce the required increase in the tidal ranges.

Owing to the flatness of the deltaic plains, the rivers in the modelled area are tidal for very considerable lengths, and it was impossible to mould geographically the whole of their tidal compartments. Beyond the limits shown in Fig. 3, Plate I, these were accordingly reproduced by labyrinthine passages, arranged to give the correct tidal volumes within manageable compass. The partitions forming each labyrinth were of sheet zinc, and were arranged to give approximately the same convergence as the banks of the river which they represented. The beds of these labyrinths were made up with sand to give the correct bed-slopes. Provision was made in the case of the Sittang labyrinth to reproduce the effects of the Alok cut.

Some care was exercised in the choice of sand for the moulding of the bed of the model, the desiderata being based on the experiments made by Professor Gibson when selecting sand for use in the model of the Severn estuary. Professor Gibson established that the size of sand for use in a river model should be about three-quarters the size of the actual sand in the river bed, the size of a sand-grain being taken as the mean of two of its axes [1]. Samples of Rangoon sand were measured microscopically to find their mean size, and suitable commercial sands were selected by testing samples in the same manner. Two sands were chosen: the first, a silver sand from Cornwall, was used for moulding the more important areas of the model, and the second, a very much cheaper ferruginous sand from the Sheffield district, was used for the remainder. These sands had practically the same density as the Rangoon sand, that is, about 2·65.

To find means of tracing the movement of the sandy bed of the model, several samples of coloured sand were also examined, but none was found suitable. At the suggestion of Mr. H. J. Collins, M.C., M.Sc., M. Inst. C.E., some of the silver sand was pigmented with powder-colour and stone preservative. It was found that the process produced a fast colour unaffected by water; subsequent tests showed that neither the size nor the density of the sand grains was measurably affected by this process, so that sand coloured in this manner could fairly be used to indicate the travel of the uncoloured sand. Some very useful results were obtained in this way.

Before starting the experimental programme it was necessary to adjust the tide-engine to give a tidal wave of the correct form. The model tidal wave could be measured on four permanent tide-gauges erected at points where such gauges were in actual fact maintained. The model-gauges were engraved on ivorine strips, and were divided into sixteenths of an inch, which corresponded to intervals of 1 foot vertically in reality. These gauges were mounted on screwed rods carried by stiff brackets bolted to the concrete portions of the model. The screwed rods permitted adjust-

[1] A. H. Gibson, "Construction and Operation of a Tidal Model of the Severn Estuary." H.M. Stationery Office, London, 1933.

ment of the gauges to show the correct readings when the water in the model was stationary and at a known level. The model-gauges were situated at :—

(*a*) Mengalun (mouth of the China Bakir river).
(*b*) Elephant Point.
(*c*) Brooking street (at Rangoon itself).
(*d*) Twante (at the junction of the Twante canal and the China Bakir river).

An additional gauge, having no counterpart in actual fact, was provided on the seaward edge of the model at a distance of about 6 miles from the main tide-plunger.

The technical officers of the port had obtained height–time curves at these gauges. Simultaneous height–time curves were obtained under varying tidal and seasonal conditions at the first three stations, and gave the necessary information regarding the true shape of the tidal wave, the rate of its progress up the Gulf, and the deformations which appeared in the wave as it entered the shallower and more constricted waters of the Rangoon river. This information was supplemented by an extensive series of predictions made by the tidal-model staff from the published harmonic constants [1].

After the model was completely moulded, similar height–time curves were obtained from the model-gauges. Readings were taken simultaneously at two gauges, each of which was watched by a separate observer, who noted the gauge reading every 5 seconds, as timed with a stop-watch by a third man. The method is laborious but reliable. The 5-second intervals represented about 45 minutes on the model time-scale, and the readings obtained were plotted on tracing paper, to the same scale as the actual height–time curves. The observations were repeated for spring and for neap tides, under monsoon and dry weather conditions, and the machinery was adjusted until the model tidal wave bore a reasonable resemblance to the real one. Figs. 8, Plate 1, show a typical comparison between model and actual time-curves.

Trial Runs of the Model.

The general programme divided the investigations into three stages. The first was a trial of the model to show that it would properly reproduce the known changes that had occurred over a given period. If this trial were successful, the second stage was to be the operation of the model, starting from present-day conditions, to find the regime which might result from the unchecked development of present tendencies. The final stage was the study in the model of any remedial measures suggested by the

[1] Footnote ([1]), p. 15.

behaviour of the model, and the selection of the most suitable course of action.

The first series of experiments was started as soon as all adjustments had been completed. The earliest available charts of sufficient accuracy and detail were those showing the surveys of 1875 and 1877, and the bed of the model was accordingly moulded to strawboard templets prepared from these charts. The model-rivers were adjusted to give average monsoon conditions, alternating with average dry-weather conditions, on the assumption that over so long a period as that contemplated for the trial runs the final result is an average result. In a similar manner the tidal cycle that was reproduced in the model took no account of abnormal variations in the actual tides.

The operating routine of the model varied little throughout the whole course of the investigations. The few important extensions and improvements in method will be explained later. During running, the model ran continuously for 24 hours each day; and the whole sequence of operations was automatically carried out by the engine and its controls. Routine tests were made several times daily to ensure that all such conditions as mean tide, salinity, etc., were correctly maintained. A watchman visited the laboratory twice nightly, and had instructions for action in the event of breakdown, or of failure in the electricity or water services. A careful log of working was maintained.

In the earlier experiments the rivers were supplied with silt differing in colour from that chosen to represent the tidal silt. Thus these two silts could be distinguished from each other at the close of a run, and the travel and distribution of silt from these two sources could be studied. As was expected, the river and tidal-silt supply-tanks did not transmit to the model all the silt which was fed into them. The residues left in each tank at the close of every run were dried and weighed, and the silt feeds adjusted to give the correct net silt-discharges.

Surveys were carried out by means of a sounding rod, marked in scale feet, which moved vertically through a sliding block, running along the top of a straight-edge marked in sea miles, divided into tenths. The large straight-edge which spanned the Gulf of Martaban was 19 feet long and was carried by the travelling gantry. A smaller straight-edge was used in surveying the rivers. The sounding rods were moved along the straight-edge by hand, and were read directly, the soundings being immediately plotted upon specially prepared master charts. This method was laborious, but was considered to give the most reliable results. In every case the model was stopped and drained slowly before survey.

Under these conditions two trial runs were made, starting from the 1875 regime, and continuing until, by the model time-scale, the 1932 stage had been reached. Each run was interrupted for the purpose of check surveys with the actual charts of 1897 and 1910. During each of these runs further comparisons were made between model and actual tidal

waves, and the velocities and directions of the model and actual tidal streams were compared by extensive float observations. The floats were timed over a 3-mile length, measured on a horizontal wooden bar held a short distance above high water by a vertical rod, rotating in a block attached to the survey straight-edge. The 3-mile gauge was rotated until it lay parallel to the line of travel of the floats, and a pointer on the vertical rod showed the bearing of the float-tracks on a protractor scale. From the position of this device on the straight-edge, and the angular reading on the protractor, it was possible to fix accurately the tracks of the floats. These model-observations were compared with meter readings taken by the Commissioners' technical staff at a number of stations in the Rangoon river and Gulf of Martaban, and were found to agree closely with them.

Each survey of the model was made by taking soundings at close intervals along the standard section-lines; these soundings were plotted directly on master charts, and contoured at intervals of 1 fathom, for easy comparison with the Admiralty and Port Commission charts. The general comparisons between model- and actual charts were good, the principal changes being demonstrated in the various model-surveys. The erodible banks of the Rangoon river, however, failed to maintain the correct side slopes, and the western extension of Middle bank was not apparent. The deterioration of the eastern channel was accelerated in the model. In addition, the ripple formation inevitable in all moveable-bed models produced very much more detail in the model-contours than was visible on the actual charts.

To overcome this latter difficulty, efforts were made to show the bed movements by measuring the volume of accretion over selected areas. A rectangle 5 miles by 2 miles was chosen, covering the scope of the Outer bar accretions. This area was plotted on all the actual charts, and on all model-surveys, after which the average sounding within the area was in each case obtained. The progressive decrease which occurred in the average sounding within this area gave a measure of the siltation. The following tabular comparison between the model- and actual results shows that a close agreement was obtained.

Comparison of Actual and Model-Siltation on Outer Bar.

Decrease of average sounding on bar area since 1875.			
	1875	1910	1932
Admiralty and Port Commission charts	0	3 feet 2 inches = 51 per cent.	6 feet 2 inches = 100 per cent.
Model-surveys in trial runs .	0	2 feet 8 inches = 44 per cent.	5 feet 3 inches = 85 per cent.

Second Series of Trial Runs.

The results of the first trial runs were on the whole encouraging; the Outer bar siltation had been faithfully reproduced and many other features had been clearly shown in the model. It was considered, however, that the method of reproducing erosion in the river-banks could be improved, and that better methods could be found for observing current-velocities and for tracing the travel of silty material. After some investigations into these points, it was decided to carry out two further trial runs in which the operation of the model would be thus modified.

Continued, and unsuccessful, attempts had been made to reproduce the natural erosion of certain stretches of the coastline. To test the erosion of various materials, a large number of separate experiments had been made, in a flume which was rocked so that water in it surged to and fro at a period equal to the tidal period of the model. Among the materials examined were a large range of sand-clay mixtures, and a number of cement–sand compositions ranging down to 1 part of cement to 300 of sand. In no case did any material satisfy the triple requirements of erosion at the correct rate, maintenance of the correct side slope, and discharge into the model of the correct amounts of material.

These experiments had dispelled any hope that it would be possible, without continuing further experiments for an inconveniently long time, to find a material which would give natural erosion in the model. During future work, therefore, the eroding portions of the bank were moulded in a heavy puddle-clay which would not, of itself, erode at all. This was cut back at frequent and regular intervals in order that the model-coastline should keep pace with the continuous recession shown on successive charts of the river. In order that the model-river should be enabled to carry away the eroded materials, and that this possible cause of the Outer bar siltation should not be omitted from the experiment, the correct quantities of sand and silt were added daily to the water in this length of the river. These quantities were based on calculation from the various charts, to show the annual volume of material carried away by the river. In order to obtain the approximate proportions of fine and coarse materials contained in the alluvial banks which were being thus eroded, site observations were conducted by Mr. W. D. Beatty, M. Inst. C.E., the Port Commissioners' Chief Engineer. These showed that the ratio by weight of the fine to the coarse materials in the eroding sections was of the order of 19 to 1; the mixture added to the model-river therefore contained 95 per cent. of silt and 5 per cent. of sand. Experiments were made to find the weight of this mixture occupying a given volume after the materials had settled and consolidated in water; from this relationship was determined the weight of the mixture which was daily added to the model.

The 3-mile gauge-length was found too short to allow accurate timing of floats, whose paths over some of the important areas were too

curved to allow the use of a longer gauge-length. Attempts to produce a suitable form of current-meter proved unsuccessful owing to the shallow depths in which this was required to work. Finally, a sheet of plate glass was obtained, 6 feet by 3 feet in size, and was mounted in a strong wooden frame, which could be supported over any area by a moveable wooden gantry. By this means the glass sheet was suspended horizontally over the model, as close as possible to high-water level. The glass was divided into $\frac{1}{4}$-mile squares by a net-work of threads on the underside and was placed over any area where current readings were desired. If a float were released so as to pass beneath the plate-glass, its successive positions could be recorded thereon at 1-second intervals by means of paint-spots: these formed a continuous track across the glass, and were subsequently replotted on squared paper to form a permanent record. The timing was done by a metronome arranged to beat at 1-second intervals. This scheme is due to Mr. Ralph Freeman, M. Inst. C.E., and worked perfectly.

The last and most important innovation was the development of chemical means whereby various silts entering the model could be traced to their final resting places, and the constituents of the silt deposits on the sea-bed allocated to their respective sources.

During the first trial runs the two differently coloured clays used for the river silt and tidal silt had become greatly intermingled, and visual tests were found useless as a means of determining the proportions in which these two silts had mixed in forming any given deposit. A search was made for some more certain means of tracing the various silty deposits in the model, and two methods were adopted after some weeks of investigation, during which the experimenters received a great deal of invaluable advice and assistance from the staff of the Department of Chemistry at University College.

The first method was the use of prussian-blue powder added to the silt in small quantities as an indicator. The powder sold commercially was found to settle through water at the correct rate, and it was also found that pure prussian blue can be easily and certainly detected in very small traces by a rapid and simple test.

The second method was an amplification of the attempts to use differently coloured clays for the river and tidal silts. Visual tests based on colour were useless, and efforts were made to discover two materials, suitable for use as model-silt, which would have similar settlement characteristics and yet have a definite and constant chemical difference. Ultimately it was found possible to obtain and use two clays each of which contained a different but constant proportion of titanium dioxide. Thanks to further co-operation by the Chemistry staff of the College, a chemical test was devised whereby the analysis of a mixture of the two silts would show the proportion in which each was present. Both methods of chemical testing were simple and reliable, and the model-staff carried out all analyses in the model-laboratory.

The third and fourth trial runs, in which the above methods of working were adopted, covered the period from 1875 to 1932. In each run the model was stopped and surveyed at the 1910 and 1932 stages, when the usual contoured surveys and volumetric checks were made.

The new method of erosion was found to have many advantages, and by maintaining proper side slopes it enabled the model to reproduce with more certainty the steady westward extension in the width of Middle bank. The silt–sand mixture added to represent eroded material was made up with the red pigmented sand, whose movement showed that this heavy material travelled to the westward around Elephant Point, landing at the eastern end of the China Bakir flats.

Very many float-track observations were made with the plate-glass apparatus. In the third run this was used to give further comparisons of the model-currents with their prototypes. In the fourth, a standard series of observations was repeated at 10-year intervals to show the changes brought about by alterations in the mouth of Rangoon river. It was found that the changes were not of sufficient importance to be regarded as having any direct influence upon the growth of the Bar.

The prussian-blue tests were used during the third run to show the deposition of silt issuing from the China Bakir river, and in the fourth run to show the travel of silt issuing from the Rangoon river. In both cases the general trend of suspended silt was in an easterly direction: the analysis of samples from the fourth run showed that little or no Rangoon river silt landed on the Outer bar. During the third run one kind of clay was introduced into the model to reproduce the silt coming from the west under tidal action, while the other was supplied to all the model-rivers. Subsequent analysis of samples from the Outer bar showed that about 70 per cent. of the bar was composed of the tidal silt. During the fourth run, the China Bakir river was disconnected from the other rivers, and separately supplied with water and with the same clay as that used for the tidal silt, and the Outer bar was afterwards found, by analysis, to contain about 85 per cent. of silt from the west. Comparison of this figure with the corresponding figure derived during the third run suggested that the China Bakir river was responsible for about 15 per cent. of the total deposit on the bar.

Examination of a large number of samples from other areas produced further evidence that the material suspended in the waters of the Rangoon river is mainly deposited on the Eastern Grove flats and in the head of the gulf. The data resulting from the use of prussian blue invariably agreed with those resulting from the titanium tests. At the close of each run, the surface of the model was completely cleared of all silt, and the upper surface of the sand was also removed, in order that the distribution of materials in one run should in no way affect the results of the next series of chemical analyses.

The proof, by the prussian-blue and titanium tests, that the bulk of the

silt travelled eastwards with the main stream of the flood tide, that the major part of the Outer bar deposits had come from the China Bakir river or from further west, and that only a small proportion of the silt leaving the Rangoon river found its way on to the bar, was considered to be one of the most valuable pieces of information obtained by the model, as there had been much controversy in the past regarding the sources of material deposited on the Outer bar and their relative importance.

At the conclusion of the four trial runs it was considered that the working of the model was satisfactory. The various comparisons had shown that the model did give a reliable representation of actual conditions, and although the trials had extended over a much longer period than was originally anticipated, they had provided opportunity for researches yielding a great deal of information of major importance in considering means for the rectification of the Outer bar.

Second Stage of Investigation: Predictive Runs.

Two predictive runs formed the next part of the experimental work. Each of these started with the model moulded to the conditions of 1932, and continued until the 1982 stage had been reached. The conditions under which the model was operated were, with two important exceptions, kept as close as possible to those governing each of the four trial runs.

In the first place a variation was made in the material representing the sea bed. The four trial runs had all started with a bed moulded in sand, which was considered most nearly to represent the conditions of the seabed in 1875. By 1932 the Port Commissioners' charts showed that large portions of the modelled area were covered with silty material some feet in thickness, and in future runs the model was moulded with a bed of this nature.

The second change concerned the erosion of the west bank of the Rangoon river, above Elephant Point. Any prediction of future erosion was undesirable, as it might be greatly affected by any hidden variation in the nature of the eroding strata. To avoid the element of doubt, erosion in the first predictive run was reproduced at about one-tenth the actual rate occurring between 1875 and 1932, and in the second predictive run this stretch of the river bank was cut back at the actual rate obtaining between 1875 and 1932. It was considered that the truth would be found somewhere between these extremes, and that comparison of the results consequent upon these two rates of erosion would show the extent to which the regime as a whole was affected by the widening of the river.

During the two predictive runs the float-track observations and the chemical silt-tracing tests were continued. The float observations completed the previous series made at 10-yearly intervals and the progressive changes observed by this method during the last trial run were found to continue. Prussian blue was applied to the Sittang, and the titanium tests

were repeated, during both predictive runs, to give an exhaustive check upon the conclusions previously drawn.

At the conclusion of the second predictive run the model was operated for a further 15 years, and the rate of introduction of silt was doubled to produce the general effects of 30 years' running. In other model-investigations it had been considered permissible to save time by increasing the effective rate of working of a model by generating spring tides only. In the Rangoon model, however, it was considered that the relatively quiescent periods during neap tides had important effects, and acceleration was carried out by the alternative described.

The results of the predictive runs, in conjunction with those previously obtained from the trial runs, led to the following conclusions:—

(1) The continuous siltation of the Gulf of Martaban is the material factor in the growth of the Outer bar.

(2) The bulk of the deposit on the Outer bar has come from the westward.

(3) The silt brought down by the Rangoon river, including that falling into it by reason of the erosion of its western bank, is of minor importance.

(4) The above-mentioned erosion has considerably straightened the lower part of the river, and the seaward end of the entrance channel is swinging to the southward, tending to cause a corresponding straightening of the bend round Elephant Point.

(5) This general straightening is merely the terminal phase of a cyclic change, and will tend to give place to a stage when further curvature will occur further seaward.

(6) The deterioration of the approaches is unlikely to continue much beyond the present stage, and further improvement will probably occur.

Study of Remedial Works.

After consideration of these points, two methods were examined whereby the limiting depth in the port entrance might be kept at a greater depth than would result from the action of natural forces alone. The first general method was the construction of training works to concentrate the discharge from Rangoon river, in order to increase the scouring tendencies over the areas where shoals were deposited. The second was the more direct method of dredging a channel through the Outer bar.

The next experiments were accordingly trials of three schemes for training walls. Two were alternative methods of closing the eastern channel, either at the upper end of Middle bank, where the eastern channel leaves the main branch of the river, or at Eastern Grove Point,

with the object of closing the eastern channel entirely and concentrating the river discharge into the western channel. The third scheme was a wall 3 miles long running seaward from Elephant Point as shown in Fig. 9, Plate 1, with the object of concentrating the main stream into the more southerly alignment that it was trying to adopt.

To save time, the effects of these walls were judged by measuring the increased velocities produced in the currents in the western channel and over the Outer bar. It was considered that, as training walls operate principally by their effects on currents, this would give all necessary information as to the efficiency of the walls, and would avoid the lengthy process of remoulding the model before trying each wall, and of running it for a considerable period in each case. This method of examination was made possible by the development by Mr. T. L. Norfolk, M. Inst. C.E., of a new type of current-meter which was used by him in the model of the Mersey estuary. The device consists of a small bead suspended in the water by a fine hair. Any motion of the water will deflect the bead from its vertical position; the amount of the horizontal deflexion bears a definite relation to the velocity of the stream, and is determined by calibration, whilst the direction of the deflexion is of course the same as the direction of flow. The deflexion is measured by a vertical sighting tube which is continually moved by hand so that the bead remains in the line of sight. As the tube carries an autographic recording pencil operated 15 times per tide by a special switch on the main engine, the pencil draws a polar graph of successive positions of the bead.

To simplify the work, the model was operated without siltation and under continuous monsoon conditions, and a series of spring- and neap-tide current readings was made at standard points on the Outer bar. The Eastern channel was then closed completely by the first training wall, and the whole series of current observations was repeated, after which the training wall was removed and the original observations checked. The routine was repeated with each of the other two training walls, and a similar series of observations was also made with the Eastern channel completely filled up, as though by natural siltation. The two walls above the eastern channel were built above high-water, but the third, from Elephant Point, was tried at various heights to see how low it could be kept. After these tests, the wall was left in position and the model operated for a short time under normal conditions and with normal siltation, in order to study its effects on the regime.

These experiments showed that in each case the effect of the training wall was entirely local, and that no effect over the Outer bar was discernible. The third wall, extending south-east from Elephant Point, was found to act as a trap for the silt travelling eastwards along the coast, and materially accelerated the advance of the China Bakir flats; the adoption of a scheme of this type would ultimately have required a continuous programme of extensions. Without considering the probable cost of such

training works it was obvious that they could serve no useful purpose in this case.

Dredging formed the subject of the next investigations. From the experimental results as a whole, it was possible to suggest a suitable location for a dredged cut through the Outer bar, arranged to take the fullest possible advantages of existing natural conditions, and of probable future tendencies. The location of this cut is shown in Fig. 9, Plate 1: 6 miles in length, it started at the end of the southern fork of Western channel, and curved gradually round until parallel to the 1910 fairway, and about 2 miles south of it, continuing in this line until deep water was reached.

This channel was tried in the model, which was first remoulded to the actual 1932 condition. The model was operated until the 1936 stage, and then an assumed dredging programme was carried out during four consecutive dry seasons. After the completion of dredging, 12 years' normal running was allowed before the model was stopped and surveyed, to measure the deterioration of the channel.

The model-channel was made equivalent to a 400-yard bed-width with side slopes of 1 in 5, and had a ruling depth of 18 feet at low water. After the first examination of the channel, a second channel of the same cross section was dredged along a line near to the first channel, but normal to the main stream of tide, and a further 12-year test run was made. Measurements were taken to show the comparative rates of siltation in the model-channel dredged with the streams and that dredged across them, to find out the degree to which a model could be used to study such matters. The channel whose alignment lay parallel to the direction of flow was found to be the more permanent, but the difference between the two channels was very much smaller in the model than would probably be the case in actual fact, since the siltation of the model cross-channel was considerably affected by the ripple formation. This was taken into account when considering the possibilities of dredging from the full-size point of view.

In the actual dredging of the model-channels, the bed-material was stirred up by a rotating templet and carried away by the flood tide. The channel was dredged gradually from the inshore end, and the strip of sea bed on the site of the proposed channel was moulded with a percentage of prussian blue, by which the material agitated by dredging was subsequently traced. It was found to have travelled in an easterly direction.

Further experiments were later carried out to test the suitability of a suggested dumping ground for dredgings removed by more usual methods. Prussian-blue powder was dumped in the area at the appropriate states of tide, and a series of samples subsequently analyzed showed the final location of the material. The site selected was abreast of the mid-point of the proposed dredged channel and was about 4 miles further seaward: the tests showed that the dredged material neither entered the Rangoon river nor drifted back into the cut.

Before and after dredging, a full series of current-observations was made at a number of stations along the line of the channel and at points in the areas on either side. These current-observations showed clearly that the model-channel had a pronounced effect in improving the currents over the Outer bar areas. The improvement was most marked in the deeper stretches of the cut, and tended to fall off as the channel slowly filled.

Measurements were made comparing the rates of siltation in the channel and over the areas on either side of it; thus it was possible to form some idea of the relative permanence of the channel. It appeared that a channel on the lines indicated would probably last for a fair time without undue maintenance, though it was not considered possible to estimate exactly the extent to which maintenance would be necessary.

Final Decisions.

With these tests the experimental work came to a close, after having been in progress for over 3 years. The final report was submitted to the Commissioners towards the end of 1935. In this it was pointed out that, whilst the port was not threatened by eventual extinction, it was evident that some limitation on the draught of vessels using the port would be inevitable unless rectification of the approaches was carried out by artificial means. The possibility of a dredging programme was outlined which, with some initial expenditure and a not excessive amount of maintenance dredging, would make reasonably certain of a satisfactory and permanent approach channel.

The report suggested that it was possibly unjustifiable for any port to incur large capital expenses to benefit a small section of its trade. Works constructed to benefit the deeper vessels must be paid for mainly out of shipping dues: thus a large proportion of traffic is penalized. Beyond a certain point increase in dues must hamper trade as much as physical obstructions, and there must be for any port an economic limit to the draught of shipping.

On receiving the final report, the Commissioners carefully reviewed the policy to be adopted for the port and its entrance channels. It was decided not to proceed with a scheme of major dredging operations over the Outer bar, and to advise shipowners that the port cannot in future deal expeditiously with vessels drawing more than 28 feet of water, so that future shipbuilding programmes can be formulated on this basis. The decision was issued to the shipping world and to the Press in a statement dated the 15th May, 1936, which has been published elsewhere. The limits prescribed for this Paper unfortunately do not allow this subject to be fully treated.

Acknowledgements.

During the course of the experiments much active help was given and many valuable suggestions were made by persons not immediately concerned with the work. The Author has to thank Professor E. G. Coker, M.A., D.Sc., F.R.S., M. Inst. C.E., and his successor, Professor G. T. R. Hill, M.C., M.Sc., and other members of the Staff of the Faculty of Engineering at University College, London. The success of the chemical investigations was very largely due to the assistance of Professor F. G. Donnan, C.B.E., M.A., LL.D., Ph.D., D.Sc., F.R.S., and Mr. Henry Terrey, B.Sc., of the Faculty of Chemistry at the same College. Advice was also obtained from the Departments of Physics and Geology.

The Author is indebted to the Commissioners for the port of Rangoon, and to Sir Alexander Gibb and Partners, for their permission to publish this Paper, and to Mr. J. Guthrie Brown, M. Inst. C.E., with whom the Author was associated in the model-experiments, for his very kind assistance in criticizing the proofs.

Mention is also due of all those who worked in the model-laboratory, and who assisted the Author throughout the course of the investigations.

The Paper is accompanied by eight sheets of drawings, from some of which Plate 1 and the Figures in the text have been prepared, and by five photographs.

Paper No. 5194.

"Schemes of Improvement for the Cheshire Dee: An Investigation by Means of Model-Experiments."†

By Jack Allen, M.Sc., Assoc. M. Inst. C.E.

TABLE OF CONTENTS.

Introduction.

The investigation described in this Paper was carried out on behalf of the River Dee Catchment Board. Its object was to study means of improving the drainage and the navigable qualities of the river Dee.

The primary trouble with this river is the continual tendency for bed-material to be transported upstream by the flood-tide, which, at Connah's Quay, occupies a time of the order of 2 hours only, as compared with 10 hours or so of ebb. A secondary, but very important, factor is the marked instability of the channel seaward of Connah's Quay. This channel is tortuous, shallow, and subject to great fluctuations in its position and configuration.

At the present time, there are in existence two training walls (*Figs. 1*). One of these, AA_1, which will be referred to as the South wall, is covered at high water, its crest-level being some 21 feet above Liverpool Bay datum *.

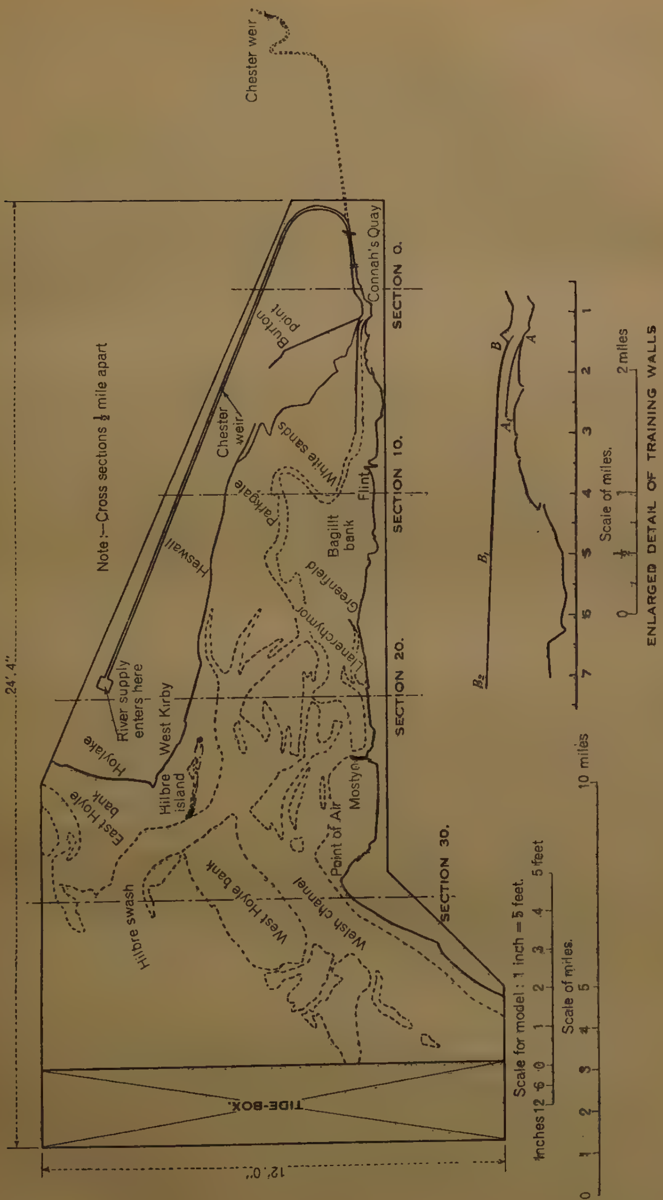
The other wall, which will be called the North wall, is above high water of spring tides over its length BB_1, and has a crest-level over its remaining portion, B_1B_2, averaging 22·0 L.B.D.

For many years, it has been thought that conditions might be improved

† Correspondence on this Paper can be accepted until the 15th August, 1939.—Sec. Inst. C.E.

* Liverpool Bay datum is 14·67 feet below O.D.

Figs. 1.



PLAN OF MAJOR MODEL.

by extending these walls, whilst the suggestion has also been advanced that a barrage situated near Connah's Quay might be used to scour the channel downstream and to prevent the travel of material upstream.

The points to be investigated may, therefore, be summarized as follows:

(1) Are the existing walls beneficial?
(2) Could they be improved by altering their crest-levels?
(3) Would an extension be desirable, and, if so, along what line?
(4) Is a barrage, equipped with locks and sluices, likely to effect an improvement?

Following the general tendency of modern practice, it was decided to study the problem by means of scale models, and accordingly two models have been constructed and operated in the Whitworth Laboratory at Manchester University, under the direction of Professor A. H. Gibson, M. Inst. C.E.

The major of these models has scales of 1 : 5,000 horizontal and 1 : 200 vertical. Its tidal period, corresponding to these scales, is 126·4 seconds, equivalent to 12·4 hours in nature, so that 1 year of tides occupies approximately 24 hours. The scope of the model is shown in *Figs. 1*, from which it will be seen that, in addition to the great expanse of sands from Liverpool bay (seaward of Hilbre island) to Connah's Quay, the canalized portion of the river Dee up to Chester weir is included along with a portion of the river extending for some miles above the weir, in fact beyond the limit of tidal action. For space considerations, the river was bent round, above Connah's Quay, by a gentle curve joining the straight portion below Chester weir. This device does not, however, invalidate the model over the region upstream of Connah's Quay, observation showing that the general behaviour is similar to that in nature. For instance, the tide-levels obtained at Chester weir relative to Connah's Quay are in virtually perfect agreement, as is the phenomenon of the bore which appears at spring tides[1]. Moreover, it has been the general practice to run pairs of comparative tests in the model; that is, to test a proposed scheme of works by operating with and without that scheme from a standard initial condition of the bed.

River water is admitted at the upper end of the model through one of a series of calibrated orifices, so arranged as to give any flow up to that of a flood equivalent to 7,500 cusecs.

For the purpose of the tests, the mean flow has been taken as 1,070 cusecs: this will be designated a "normal" river.

Tides are produced by the displacement of a steel plunger situated at the seaward end of the model and driven through an epicyclic train of gear-wheels by a constant-speed motor. The epicyclic gearing automatically varies the stroke of the plunger from a maximum at spring tides to a minimum (fourteen tides later) at neaps, and back to a maximum in a further fourteen tides, for the next springs.

[1] The bore has been discussed in some detail in a previous Paper by the Author "Experiments on Water Waves of Translation in Small Channels." Phil. Mag. Series 7, vol. xxv (1938), p. 754.

The crank-pin attached to the last wheel of the epicyclic train rotates in a brass block which itself slides along a slotted bar. This bar, pivoted at one end, serves to produce a longer time of rise for the plunger than fall, or a longer ebb than flood, as is demanded by the shape of the tide-curve in Liverpool bay.

The varying thrust of water on the plunger is balanced by means of a weight suspended from a cam of appropriate shape.

The bed-material used to form the sand-banks in the model is a sand having a mean grain size of 0·0071 inch and a mean ratio of longest to shortest diameters for its individual grains of 1·53. In certain tests, however, comparative results were obtained with a coarser sand (mean diameter 0·0092 inch, mean ratio 1·58); in others, with powdered pumice (mean diameter 0·0115 inch, ratio of longest to shortest diameters of individual grains 1·58). The specific gravity of the sand was 2·63, and of the pumice 2·00.

Since the river water of the Dee is, for practical purposes, clear, no silt in suspension was supplied in the river water of the model.

The other tidal model used in this investigation has scales of 1 : 40,000 horizontal and 1 : 400 vertical, its tidal period therefore being 22·3 seconds. Although this model is only some 4 feet in length, it has proved to be of considerable assistance. Tidal heights are found to be reproduced in it with considerable accuracy, the same principle of displacement by a plunger being adopted as in the large model. Preliminary experiments made in the small model provided a useful guide as to schemes which might profitably be tried on the major model, and indeed the effect on tidal phenomena of various training walls was sensibly the same in the two models. It is to be understood, therefore, that although the experiments described in this Paper were made, unless specifically stated to the contrary, on the larger model, at the same time many of the results were confirmed by the smaller model. Further, no material discrepancy appeared in the conclusions reached from the two models during any of the tests, which were carried out in both models.

Tide-levels in the larger model were read chiefly by means of pointer gauges equipped with verniers reading directly to 0·01 inch, but partly on graduated scales supported by suitable brackets. Measurements of tide-levels in the smaller model were, however, facilitated by pointer gauges attached to micrometer screws with verniers capable of a direct reading to 0·001 inch.

The methods adopted for moulding and surveying were as described by Professor Gibson in the Vernon-Harcourt Lecture, 1935–36*.

* "Tidal and River Models." Journal Inst. C.E., vol. 3 (1935–36), p. 699. (October Supplement, 1936.)

The Investigation.

Preliminary.

Existing charts of the Dee were supplemented by a series of aerial photographs taken at L.W.O.S.T. on the 21st June, 1937. These photographs were resolved into a scale plan of the low-water channels, through a method devised by a colleague of the Author's, Mr. J. L. Matheson, M.Sc., Assoc. M. Inst. C.E., and described by him in a Paper already published by The Institution[1]. The bed of the model having been moulded to agree with available information, the first tests were devoted to the adjustment of the plunger in order to generate the correct tide at the seaward end of the model. It will be appreciated that, while it is generally possible to achieve a reasonable first approximation to the size and shape of plunger by design-calculations based on the estimated volumes of water entering and leaving the estuary at various stages of the tide, yet the phenomena may, in a particular case, be so complex as to subject the designed plunger to modification by experiment. The plunger of the Dee model was, in fact, so modified by means of additional tapered portions at its end and along its base, until the rate of rise and fall at Hilbre island was in sensible agreement with the tide-curves observed in the natural estuary. *Figs. 2* demonstrate the agreement obtained. A comparison of the tide-curves at Connah's Quay in the natural estuary and in the model is not so readily obtained. In the first place, the rate of rise of the tide at Connah's Quay, in nature, is known to vary considerably from time to time, according to the configuration of the estuary, the state of the river, and the prevailing meteorological conditions. Thus, in discussing a Paper read by Mr. F. Webster, M.C., M. Eng., Assoc. M. Inst. C.E., before the Liverpool Engineering Society in November, 1929, Mr. A. Caradoc Williams, Assoc. M. Inst. C.E., stated that "the flow at Connah's Quay lasts for a period of from 2 hours 15 minutes down to 1 hour 50 minutes. . . . During neap-tides there may be no rise at all at Chester, and in one case recently there was only a three-foot rise at Connah's Quay." (The neap-range at Connah's Quay is frequently 7·5 feet, and at Chester weir 2·5 feet.)

The model, in fact, reveals the interesting phenomenon that high-water level at Connah's Quay, for a given tide at Hilbre, depends essentially upon the level of the Bagillt bank. By lowering this bank it is readily possible to raise high-water level at Connah's Quay by as much as 17 inches.

In *Figs. 2* it will be observed that two tide-curves are shown, taken at different times and with different low-water levels, at Connah's Quay in the model, whilst, for comparison, an estuary-curve for the same place is also given.

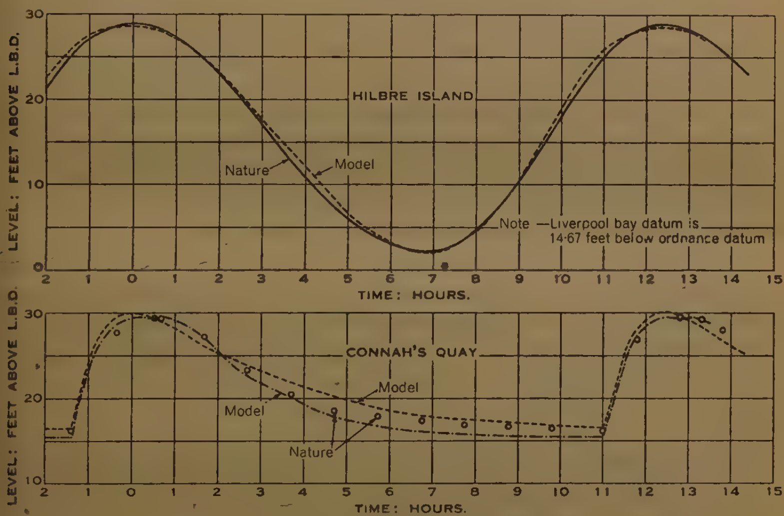
[1] "An Aerial Survey of the Estuary of the River Dee, Employing a Simple Method of Rectifying Oblique Photographs." Journal Inst. C.E., vol. 10 (1938–39), p. 47 (November 1938.)

The comparison thus afforded appears to justify the conclusion that the tide at Connah's Quay is sufficiently well represented in the model, if not in fact, that were it practicable to reproduce in the model the identical conditions obtaining at the time of any particular observations made in the natural estuary, the agreement would be within the limits of observation.

It is interesting to note also that low-water level at Connah's Quay is somewhat lower, in the model, at neaps than at springs (for a given river-discharge), and this phenomenon is believed to be in agreement with nature.

A series of tests was next made to study the effect of various modifica-

Figs. 2.



ACTUAL AND MODEL TIDE-CURVES.

tions of the existing training walls on the spring-tide levels of high water and the times of ebb and flood at Connah's Quay and Chester weir. It was assumed that any alteration which prolongs the flood and shortens the duration of the ebb-tide will tend to improve the condition of the river between Connah's Quay and Chester. Since, however, almost the whole of the movement of the bed occurs during the first half of the flood and the second half of the ebb, observations were made of these times also. Twenty experiments were carried out on these lines, care being taken to adjust the moulding of the bed from time to time so as to preserve comparable conditions. In particular, the level of low water was kept constant.

These tests showed that any changes in the North wall alone, whether in the way of lengthening or shortening or of raising or lowering, are, on

the whole, detrimental. For example, raising the North wall above high water throughout its length reduced the time taken by the first half of the flood-tide by some 6 per cent.; high water at Connah's Quay was raised 0·1 foot. Removing the low part of the North wall raised high water at Connah's Quay by 0·3 foot and high water at Chester weir by 0·1 foot, at the same time reducing the time of the first half of the flood at Connah's Quay by as much as 25 per cent.

Raising the South wall above high-water level, however, showed a definite gain, especially when such a raised wall was also extended to a point opposite the end of the high part of the North wall. This latter scheme had the effect of increasing the time of half flood at Connah's Quay by 30, 25, and 16 per cent., according to whether "no river", "normal river", or "flood river" respectively was in operation. At the same time, high-water levels at Connah's Quay and Chester weir were lowered some 4 inches, an effect which, from the point of view of drainage, would be beneficial. Moreover, the scouring effect of the ebb-tide was improved, since the time of the last eighth of the ebb at Connah's Quay was reduced by 5, 1, and 10 per cent. for no river, normal river, and flood river respectively.

A slight additional improvement might be effected by extending the South wall, as a half-tide wall, to the end of the present low North wall, but the gain would not appear to justify the extra cost.

When the existing training walls were taken out of the model, high-water levels at Connah's Quay and Chester weir rose 6 and 5 inches respectively; the time of the first half of the flood at Connah's Quay was reduced by 37 per cent. It is established, therefore, that the existing walls are of value from the point of view of drainage and of decreasing the volume of bed-material transported upstream.

Schemes for opening a Channel through the Bagillt Bank.

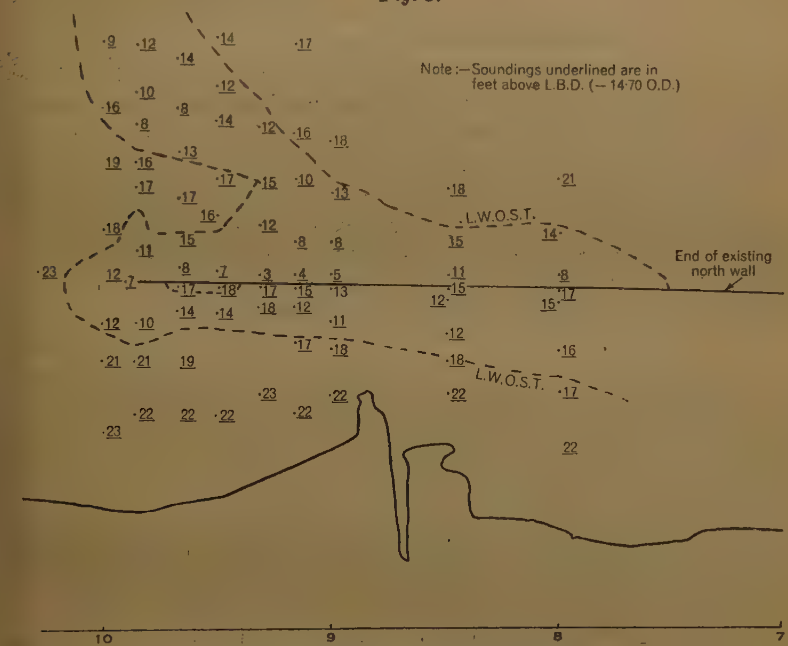
(*a*) The North wall being extended in stages for a distance of approximately 1½ mile did not lead to the formation of a channel through the Bagillt bank. The scour was felt for a distance of about 400 feet beyond the toe of the wall, but the ebb then doubled back behind the wall to find its way into the existing channel. Again, it was found that more material was carried upstream on the flood than was ultimately removed by ebb-scour. For this reason, the low-water level at Connah's Quay rose 3·3 feet during a run of a hundred spring tides, at the end of which the configuration of the channel was as shown in *Fig. 3*. Substantially the same results were obtained using a bed-material of powdered pumice, and again with a coarser grade of sand.

(*b*) Even with a South wall extended above high-water level as far as the end of the existing high North wall, and the North wall continued as far as section 11 (that is, 2 miles beyond its present limit), a channel was not produced through the Bagillt bank. This statement applies whether the North wall is extended as a low or a high wall. During a run of three

hundred and eighty-nine spring tides with a high extended North wall, low water at Connah's Quay rose 2·3 feet, high water fell 1·6 foot, and at Chester weir high water fell 1·0 foot.

(*c*) With the North wall extended to section 11 and a channel dredged through the Bagillt bank alongside the wall and beyond it, to join the existing channel at Llanerchymor, it was found that the channel deteriorated. In particular, the dredged channel seaward of the end of the wall showed a marked tendency to diverge from the straight and to scour to the southward.

Fig. 3.



MODEL AFTER 104 SPRING TIDES.

(*d*) The next test was carried out to determine whether, with the estuary and training walls in their present state, a straight channel dredged through the Bagillt bank would be maintained. This dredged channel had a bottom width of 200 feet, was in a line forming a prolongation of the existing trained channel, had a bed-level of 14·00 L.B.D., and involved the excavation of 1·3 million cubic yards. The dredged channel was found to deteriorate rapidly, even when the North wall was extended at half-tide level as far as section $10\frac{1}{2}$.

(*e*) Tests were also made, with the full spring–neap cycle of tides, on a completely trained channel through the Bagillt bank, bounded by two walls in the form of a continuation of the existing walls as far as section $15\frac{1}{2}$.

This amounted to extending the North wall 7,000 yards and the South wall 11,000 yards. The width between the walls varied from 410 feet off Flint to 750 feet at the mouth of the walls. A channel was dredged between the walls to give the same low-water level as with the existing channel, and it was found that high water at Connah's Quay was lowered by 0·7 foot, while the rate of rise of the tide there was very considerably reduced.

Unfortunately, however, a strong ebb-current (about 7·4 knots) was created, impinging on the foreshore seaward of the walls and resulting in pronounced scour, the scoured material being deposited in the lower channels.

A strong cross current occurred, around spring tides, sweeping over the tops of the walls, almost normal to the trained channel, between sections 5 and 9. Not only is this objectionable from the point of view of navigation, but it caused a scour to a depth of between 10 and 14 feet behind the South wall in the region of sections $5\frac{1}{2}$ and $8\frac{1}{2}$. This scoured material was carried upstream behind the walls as far as section 4, where the greater portion of it was swept over the South wall and deposited in the channel, causing a shoaling of as much as 6 feet in 1 year of tides.

In order to prevent this action, it was found necessary to raise the South wall to high-water level everywhere upstream of section 8, after which some 680,000 cubic yards of sand were dredged from the channel between Connah's Quay and Flint. As a result of this dredging, low water at Connah's Quay was observed to be 16·10 L.B.D. During a further 4·75 years of tides, however, the channel again deteriorated to such an extent that low water at Connah's Quay was found to be 17·40 L.B.D., and an additional 715,000 cubic yards had to be removed from the bed of the channel in order to restore the low-water level.

A gap was next made in the South wall opposite Flint Tips. The width of the gap was 420 feet, and a channel was dredged between it and Flint and revetted with walls having a crest-level of 16·00 L.B.D. It was found, however, that, about 1·55 hour after the beginning of the flood at the mouth of the walls, a strong cross current of as much as 5 knots swept diagonally across the gap.

After running for a total period of 9·3 years, a survey was taken, and is shown in Fig. 4, Plate 2. For comparison, the initial condition of the bed is shown in Fig. 5, Plate 2. During the experiment, there was considerable deterioration of the channel between the mouth of the walls and the Mostyn deeps. An appreciable bore was observed to travel up the trained channel on the flood of spring tides.

Following the 9·3-year survey, the channel between the walls was again dredged and a low-water level of 15·40 L.B.D. obtained at Connah's Quay. Detailed observations of currents were then made and revealed the persistence of flood-tide cross currents proceeding towards the Welsh coast, at right angles to the walls, over the whole length of the North wall.

These currents were especially evident in the region of sections 9 and 12, where they attained a speed of 6 knots. It was clear, therefore, that in order to facilitate navigation it would be necessary to raise the North wall above high-water level almost throughout its length, and whilst this device was studied in some detail, the scheme of a trained channel through the Bagillt bank could not be freed of certain inherent faults, viz:

(i) The amount of dredging to establish and maintain a navigable channel is large.

(ii) The channel, being straight, is essentially less stable than a curved channel.

(iii) The ebb-current seaward of the toe of the walls is rapid and would erode any foreshore not heavily protected.

(iv) The approach to the mouth of the walls from the Hilbre side of the estuary is difficult.

(*f*)—Tests on the smaller model with bed initially level. In order to determine what sort of configuration of sand-banks would be produced by the action of the tides if the bed of the estuary were initially level, four experiments, one with sand and three with pumice as bed-material, were carried out on the smaller model. For two of these tests, one with sand and one with pumice, all training walls were removed; in the other tests, a wall representing the existing North wall was introduced. The results are shown in *Figs. 6* (pp. 40, 41).

They are of interest in demonstrating that, without training walls, the main channel tended to make towards the Wirral side of the estuary; nor did the introduction of the training wall materially affect this tendency beyond the toe of the wall.

In every case the Bagillt bank was developed as a prominent feature of the estuary. It is produced partly by the direct transportation of the flood-current and partly by an eddy contained in the flood-tide and making towards the Welsh shore.

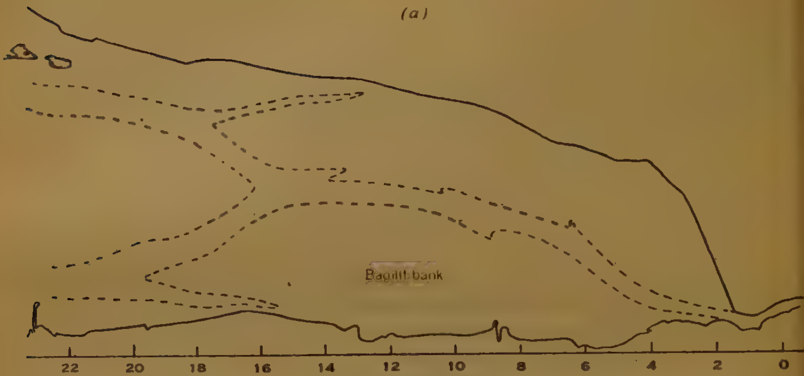
Schemes for a Trained Channel other than through the Bagillt Bank.

Experiments have been made on numerous schemes of single and double training walls and groynes. These indicate that a trained channel between two walls offers the only solution. They also show that for such a scheme to be successful, the walls would have to be extended for a considerable distance down the estuary to a point where the channels are comparatively stable.

Among the many schemes tried were those shown in *Figs. 7* (pp. 42, 43). Any of these schemes reduces the rate of rise of the tide at Connah's Quay, especially the early portion of the flood, and tends therefore to minimize siltation in the upper stretches of the river. From this point of view, scheme B is preferable to scheme A, and it also ensures the reclamation of a considerable area of the White sands. The cross wall joining the

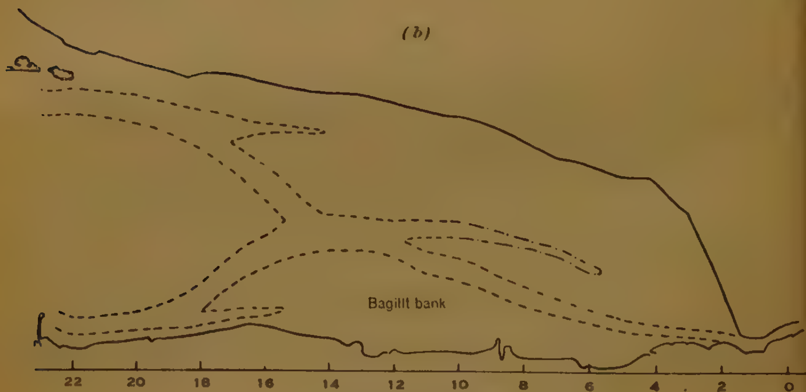
Figs. 6.

(a)



BED MATERIAL SAND.
4,300 TIDES.
NO TRAINING WALLS.

(b)



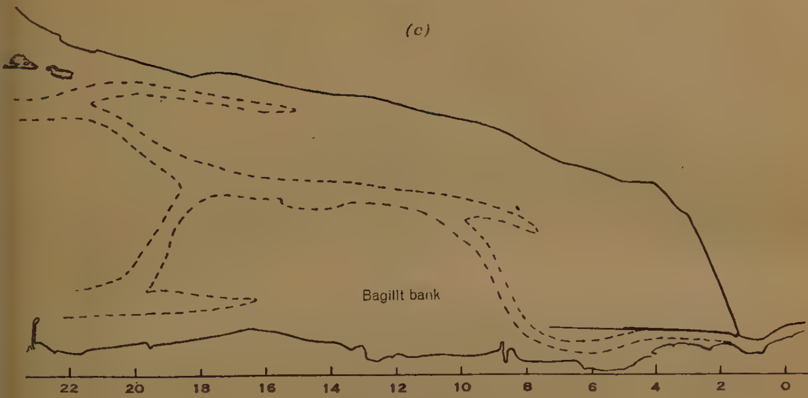
BED MATERIAL PUMICE.
1,000 TIDES.
NO TRAINING WALLS.

South wall to the Flintshire foreshore was found desirable to protect the foreshore against the scouring effect of the flood-tide, which rushes into the funnel-shaped basin between the South wall and the shore.

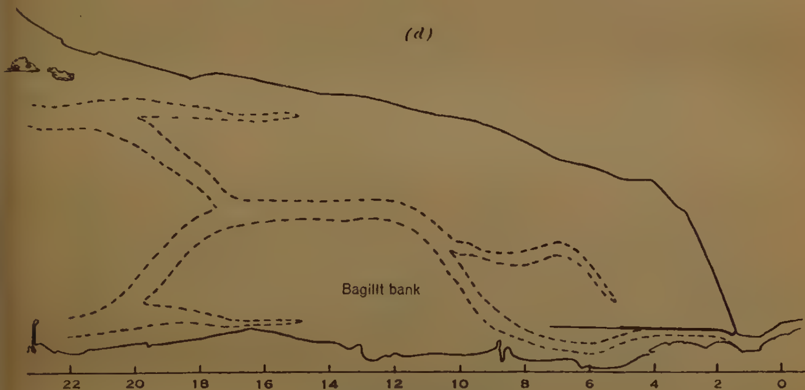
Scheme B, however, still leaves the channel seaward of section 9 in an unsatisfactory unstable condition. This objection could be met by extending the walls as in scheme C.

Scheme D compares favourably with scheme C in that the trained channel is more readily accessible from both sides of the estuary, and the South wall runs closer to the harbour at Flint, so that a gap may be left in that wall to communicate with the harbour by a revetted channel. By reclaiming the foreshore between there and Connah's Quay with the

Figs. 6.



BED MATERIAL PUMICE.
1,200 TIDES.
NORTH TRAINING WALL.



BED MATERIAL PUMICE.
4,000 TIDES.
NORTH TRAINING WALL.

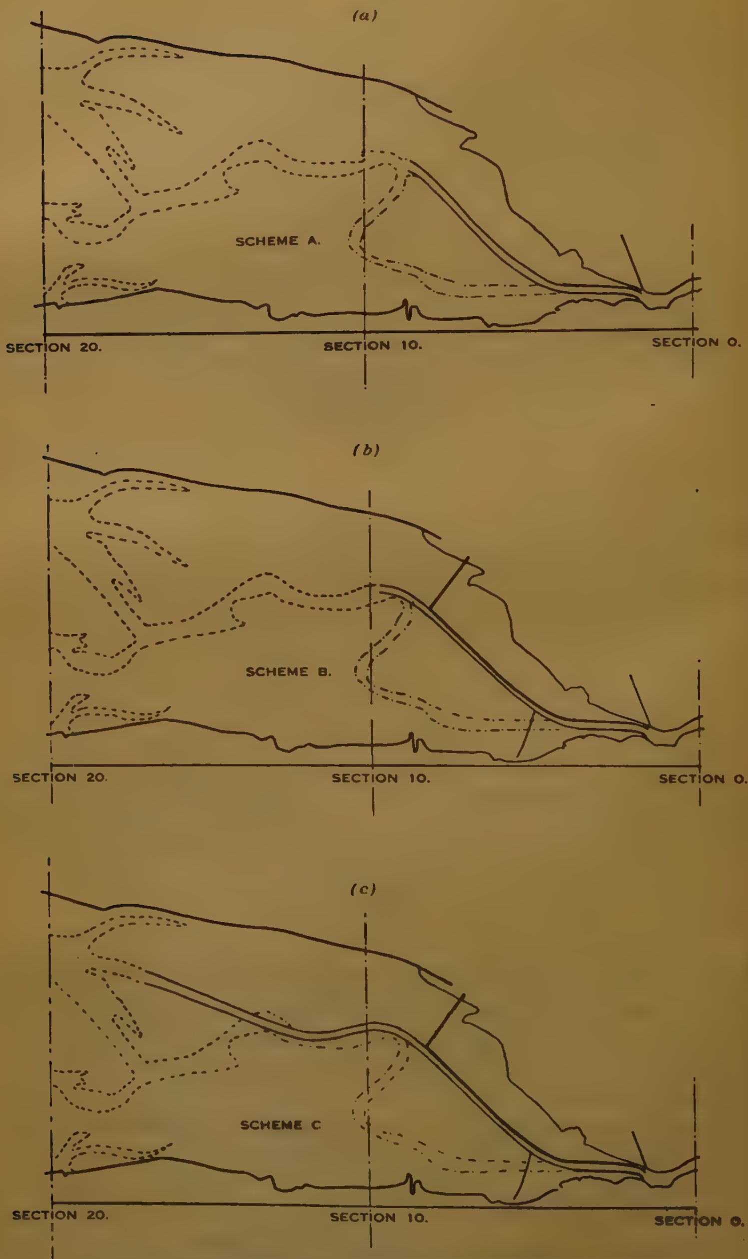
aid of a wall above high-water level, as in scheme E, the effective flood-scouring velocity was slightly reduced, and a material gain secured in the easier access afforded to Flint harbour. The high South wall running into the foreshore prevents the marked sweep of the flood-tide across the revetted channel.

Scheme E accordingly possesses the balance of advantages of all the schemes tested.

Experiments on Scheme D.

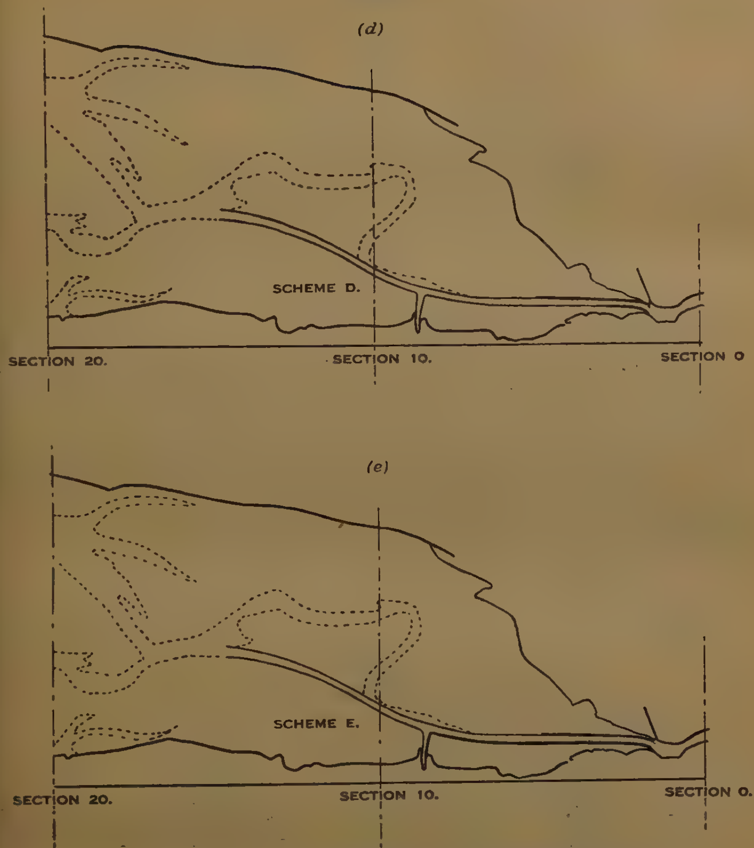
The distance between the walls in this scheme is approximately 420 feet, from Connah's Quay to section 12, below which there is a gradual

Figs. 7.



NOTE.—Walls shown in thick lines are above H.W.S.T.

Figs. 7.



NOTE.—Walls shown in thick lines are above H.W.S.T.

widening to 615 feet at the mouth of the walls. Experiment indicates that, if the gap between the walls is gradually increased from 420 feet at section 10 to 1,020 feet at the mouth, maximum flood-velocities between the walls are increased by some 10 per cent., and the high-water levels at Connah's Quay and Chester weir are lowered 0·2 and 0·3 foot respectively, whilst low-water level is sensibly unaltered.

The walls of this scheme lie parallel to the direction of flood and ebb during the major portion of the tide, whilst with walls of crest-level as given in Table I (p. 44) no objectionable cross currents were detected:

The length of extension of the North wall is 7,000 yards and that of the South wall 11,000 yards, the distance from Connah's Quay to the mouth of the walls being 7·1 miles, or 1·5 mile less than with the walls of scheme C.

A channel was dredged between the walls of this scheme and a test of

TABLE I.

	Section.								
	6	7	8	9	10	11	12	13	Mouth.
Crest-level: feet above L.B.D.									
North wall . .	24	24	25	25	25	22	22	18	12
South wall . .	H.W.*	H.W.*	H.W.*	26	23	22	22	18	12

* South wall in scheme E. In scheme D, which, as previously explained, is not so satisfactory, these crest-levels of the south wall were 23·0 L.B.D.

9·3 years of tides made in order to investigate the behaviour of such a trained channel and of the channels upstream and downstream of the walls. For comparison, a run of 9·3 years was also made with existing walls only. Surveys taken showed that the dredged channel was well maintained; the bed above Connah's Quay was appreciably lower with the extended walls than with existing walls, and below the seaward end of the extended walls the channels suffered no deterioration compared with the test made on existing walls.

Spring-tide observations made at the end of the 9·3 years' test showed the following comparative results:

(+ sign signifies higher level, or longer time, with scheme D than with existing walls. " Turn of surface " means reversal of direction of surface-current from flood to ebb.)

(*a*) *With "No River" Flow:*

High-water level at Connah's Quay:	−0·7 foot
Low-water ,, ,, ,, :	−1·8 feet
High-water level at Chester weir:	−0·7 foot
Low-water ,, ,, ,, :	−3·0 feet

Time from beginning of flood at Connah's Quay to:

" Half-tide " at Connah's Quay:	+300 per cent.
" Turn of surface " ,, :	+47 ,,
Beginning of flood at Chester weir:	+54 ,,
" Turn of surface " ,, ,, :	−2 ,,

(*b*) *With "Normal River" Flow:*

High-water level at Connah's Quay:	−0·7 foot
Low-water ,, ,, ,, :	−2·7 feet
High-water level at Chester weir:	−0·2 foot
Low-water ,, ,, ,, :	−3·6 feet

Time from beginning of flood at Connah's Quay to:

" Half-tide " at Connah's Quay:	+237 per cent.
" Turn of surface " ,, :	+36 ,,
Beginning of flood at Chester weir:	+41 ,,
" Turn of surface " ,, ,, :	+42 ,,

(*c*) *With "Flood River" Flow:*

High-water level at Connah's Quay:	−0·3 foot
Low-water ,, ,, ,, :	−2·2 feet
High-water level at Chester weir:	−0·3 foot
Low-water ,, ,, ,, :	−3·0 feet

Time from beginning of flood at Connah's Quay to :

" Half-tide " at Connah's Quay :	+310 per cent.
" Turn of surface " ,, :	+39 ,,
Beginning of flood at Chester weir :	+42 ,,
" Turn of surface " ,, ,, :	+35 ,,

At Connah's Quay with the walls of scheme D, the flood begins 36, 41·5, and 28 minutes earlier than with the existing walls, according as to whether there is " no river", normal flow, or flood discharge respectively.

Experiments on Scheme E.

(A) Immediately following the test on scheme D, the walls were modified to the layout of scheme E by reclaiming the foreshore between Connah's Quay and Flint.

The information tabulated below was then obtained :

(— sign indicates water-level lower with scheme E than with existing walls after 9·3 years. Readings marked (1) are for " no river " flow, (2) for " normal river " flow, and (3) for " flood river " flow.)

Connah's Quay, high-water level (1) :	—0·4 foot.
,, ,, ,, (2) :	—0·5 ,,
,, ,, ,, (3) :	—0·2 ,,
Connah's Quay, low-water level (1) :	—2·6 feet.
,, ,, ,, (2) :	—2·9 ,,
,, ,, ,, (3) :	—2·7 ,,
Chester weir, high-water level (1) :	—0·8 foot.
,, ,, ,, (2) :	—0·4 ,,
,, ,, ,, (3) :	—0·6 ,,
Chester weir, low-water level (1) :	—2·6 feet.
,, ,, ,, (2) :	—3·0 ,,
,, ,, ,, (3) :	—2·3 ,,

With scheme E, the maximum surface-flood velocity near Connah's Quay is about 0·90 of that measured with existing walls after a run of 9·3 years. During the first half-hour of the flood, however, the velocity with scheme E is little more than half that experienced with existing walls. Again, velocities measured late in the ebb are some 20 per cent. greater with scheme E.

In the experiments discussed above, the soundings in the dredged channel between the walls were as follows :

TABLE II.

Section :	2	3	4	5	6	7	8	10	12	14	Mouth.
Bed-level: feet above L.B.D. .	9	12 (10)	8 (8)	8 (7)	8 (4)	7 (4)	9 (1)	7 (1)	3 (—2)	2 (—3)	—3 (—3)

If the bed be lowered to the levels shown above in brackets, the comparative spring-tide levels become:

	Normal river.	*Flood river.*
Connah's Quay, high water:	−0·3 foot	−0·3 foot
,, ,, low water:	−3·1 feet	−2·7 feet
Chester weir, high water:	+0·1 foot	−0·6 foot
,, ,, low water:	−3·0 feet	−2·3 feet

(The figures given above are relative to existing walls after 9·3 years.)

The effect of lowering the bed is greatly to reduce the maximum flood-velocities between the walls and above Connah's Quay, without appreciably affecting the ebb-velocities.

With "normal river" flow, the spring high-water level at the mouth of the walls is 0·1 foot, and the low-water level is 1·0 foot, lower than with the existing walls.

(B) A complete test of scheme E has been performed in the following manner:

The bed of the model was completely remoulded to the standard initial condition. Starting with existing walls, extensions were made as in the programme below:

(*a*) After 1·7 month of tides, the South wall was raised above high water and was extended as a high wall as far as section 7.

(*b*) After 11·5 months, the walls were extended to section 8 (the South as a high wall; the North as a low wall).

(*c*) After 1·8 year, the foreshore was reclaimed as far as Flint Tip with a high wall.

(*d*) After 1·92 year, the North wall was extended to section 9, and the South wall to section 10, leaving a revetted gap at Flint.

(*e*) After 2·64 years, the South wall was extended to section 12, and a North wall inserted between sections 11 and 12, leaving a gap in the North wall between sections 9 and 11.

(*f*) After 2·98 years, the walls were extended to section 13½, and 400,000 cubic yards were dredged from between these extended walls.

(*g*) After 3·76 years, the walls were carried to their full extent and 450,000 cubic yards of material were dredged.

(*h*) After 4·66 years, the scheme was completed by building a wall across the gap previously left in the North wall between sections 9 and 11. 3,750,000 cubic yards of material were dredged between the walls to give the following channel-bed levels.

TABLE III.

Section:	2	4	6	8	10	12	14	Mouth.
Channel-level: feet above L.B.D.	10·5	10	9	9·5	7	2	1	−2

Of the total 4,500,000 cubic yards dredged, the first 850,000 cubic yards was deposited behind the South wall; the remainder was deposited partly behind the South and partly behind the North. Dredging was carried out over the full width between walls.

(*j*) After 14·3 years, or 9·64 years after completing the scheme, a full survey was made, a partial reproduction of which is shown in Fig. 8, Plate 2.

Following the test described above, the model was remoulded to the standard initial conditions, and a run of 14·3 years made with existing walls only.

Comparable tide-data observed at the conclusion of the tests are set out below:

TABLE IV.—SCHEME E: EFFECT AFTER 14·3 YEARS (NORMAL RIVER).

(A negative sign indicates a lower water-level with scheme E than with existing walls.)

	Spring tides.	Neap tides.
Connah's Quay, high-water level . .	−0·4 foot	−0·6 foot
,, ,, low-water level . .	−2·8 feet	−3·2 feet
Chester weir, high-water level . .	−0·2 foot	*
,, ,, low-water level . . .	−4·6 feet	−4·5 feet

* With scheme E, there was a tidal range of 2·7 feet at Chester weir; siltation during the 14·3 years of normal river with existing walls caused the neap tide to vanish below Chester weir.

On spring tides, the effect of scheme E was to increase the time from the beginning of the flood to high water at Connah's Quay by more than 100 per cent.; relative to low water at Hilbre island, the flood-wave reached Connah's Quay some 27 minutes earlier than with existing walls. The corresponding figures for neap tides are 54 per cent. and 60 minutes.

Between Connah's Quay and Chester weir, the bed-level was very appreciably higher with existing walls than with scheme E. An especially significant point is that, over the region from 1½ to 2 miles upstream of Connah's Quay, the bed rose from 13·50 L.B.D. to 14·40 L.B.D. during the period of construction of scheme E; afterwards the bed was scoured to 12·30 L.B.D.

In the 9·64 years after the completion of the scheme, the bed of the channel between the walls rose about 1 foot from its dredged state, but there was every appearance of stability at the self-adjusted level. Accompanying the natural readjustment of the bed below Chester weir, the level of low water at Connah's Quay was observed to rise 0·4 foot and at Chester weir to fall 0·7 foot during the period of 9·64 years.

Over the region of the estuary seaward of the mouth of the walls, no deterioration was observed as compared with the test on existing walls.

Tests on the Effect of a Suggested Barrage below Connah's Quay.

It has been suggested that a barrage, equipped with locks, might be built across the channel at the end of the existing South wall. This barrage would be provided with sluice gates to open during the latter part of the ebb tide, discharging the water accumulated upstream during the period of closure. No tide water would be allowed to pass above the barrage on the flood, and the scheme offers the attractive prospect of preventing the drift of sand up the river while encouraging scour of the lower channel on the ebb.

Accordingly, a model-barrage was introduced, containing a gate which was opened by a solenoid operated from the main driving mechanism in a manner similar to that described by Professor Gibson[1].

The period of opening of the gate was varied with varying river flow to give a minimum water-level of +6 feet O.D. above Connah's Quay. With "normal river" flow, the necessary time of opening was 2·15 hours, and with a flood river of 4 hours the level of the sluice gate-sills was −6·00 O.D.

A pair of corresponding tests was carried out:

(*a*) with existing walls, except that the South wall was raised above high-water level;

(*b*) with the barrage in operation.

In each case, the test started from the standard initial bed-condition below the site of the barrage. In case (*a*) the bed conditions were also standard above the barrage, but in (*b*) the bed above Connah's Quay was moulded to a uniform level of 8·7 L.B.D. (that is, 6 feet below O.D.).

Each run lasted for 1 year of tides, during which the river flow was varied as follows:—

3 months	"no river" flow
2 ,,	"normal river" flow
1 ,,	"flood river" flow
2 ,,	"normal river" flow
1 ,,	"flood river" flow
3 ,,	"normal river" flow

The comparative results thus obtained showed that, whilst there was considerable scour immediately downstream of the gate, the scoured material was largely redeposited in the channel upstream of Flint, and that the average bed-level between sections 4 and 8½ was raised by as much as 2 feet. Between sections 8½ and 17, the average level was raised some 12 inches. During an additional year's test on the barrage, further deterioration was observed, especially between Flint and the barrage. There was no indication that, as was hoped, a channel of level 6 feet below O.D. could be obtained, except by dredging. Further, such a dredged

[1] "Construction and Operation of a Tidal Model of the Severn Estuary", Appendix A. H.M. Stationery Office, 63–78–2, London, 1933.

channel would not be maintained, for during periods of little river flow, when the volume of water available for discharge through the sluices is small, fresh material would be carried up to the barrage on the flood tide. This general effect is demonstrated by an experiment made with a closed barrage and with no river-discharge. *Figs. 9* (p. 50) show the state of the channel after one hundred and seventy-three spring tides with and without the barrage, and also show the channel after a total of eight hundred and one tides with a closed barrage; the silting of the channel due to the suppression of the ebb discharge from the upper river is shown graphically.

Tide observations showed that the level of high water just below the barrage is sensibly unchanged; low-water level is, however, directly affected by the changed level of the bed. Under flood-river conditions, the sluice gate discharged for a short time under a head of 12·5 feet. This involves a spouting velocity of some 28 feet per second. The head could, of course, be reduced by opening the gates earlier; thus if they were opened 8 hours before low water, the maximum head would be reduced to about 4·5 feet, and the spouting velocity correspondingly to some 17 feet per second, but the ebb scour would become less effective.

Conclusions.

The main results obtained from this investigation of the Cheshire Dee may be summarized as follows :—

(*a*) An improvement in the channel upstream of Connah's Quay is anticipated if the present South wall were raised above high-water level and were continued as such as far as the end of the existing high North wall.

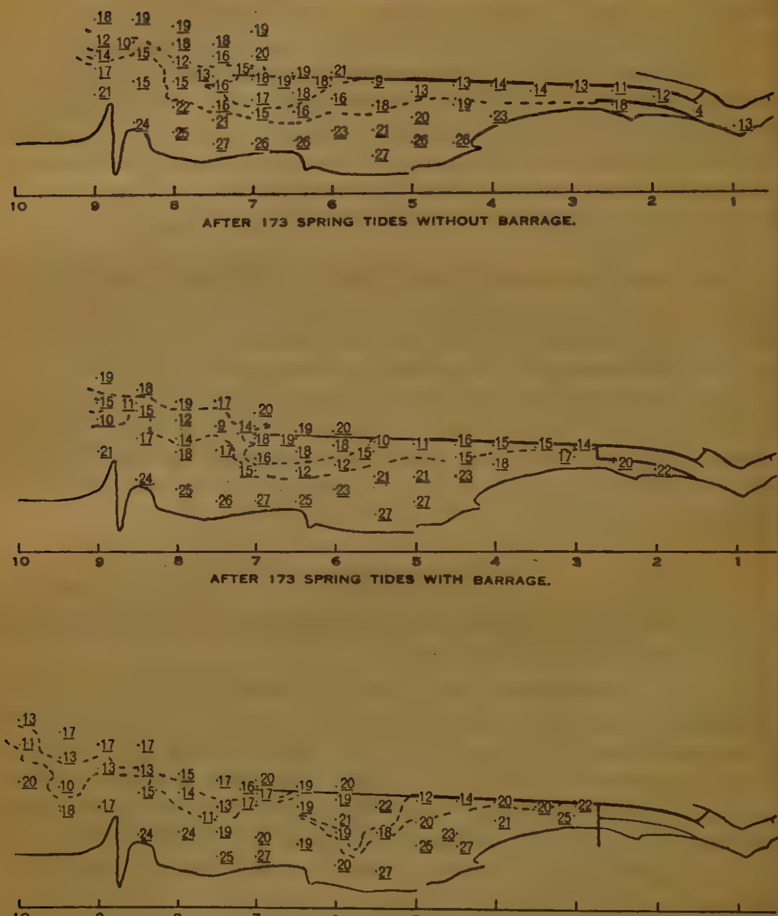
(*b*) In order to improve the channels below Connah's Quay, whether for the sake of drainage or of navigation, some system of training walls extending for a considerable distance beyond the present walls is necessary.

(*c*) A channel trained between double walls is the only satisfactory solution.

(*d*) As described in the section entitled "Schemes for opening a channel through the Bagillt bank" (p. 36), the possibility of training a channel through the Bagillt bank into Mostyn deep by means of extensions to the existing walls in continuation of their present line, has been explored in some detail. The scheme is only practicable by the use of a very costly system of walls; it has many disadvantages, as set out in sub-section (*e*), p. 37.

(*e*) The scheme offering the balance of advantages of the many which have been studied is that designated scheme E in *Figs. 7* (pp. 42, 43). This scheme would provide a total length of trained channel seaward of Connah's Quay of 7·1 miles. It would involve an extension of 11,000 yards to the South wall and of 7,000 yards to the North wall. The channel thus formed would pass within 500 yards of Flint Tips and a gap could be left in the South

Figs. 9.



AFTER 173 SPRING TIDES WITHOUT BARRAGE.

AFTER 173 SPRING TIDES WITH BARRAGE.

AFTER 801 SPRING TIDES WITH CLOSED BARRAGE.

Note:—Figures underlined represent feet above L.B.D.
L.W.O.S.T. indicated thus ------------

wall opposite Flint to provide access to Flint. Provided that the foreshore between Connah's Quay and Flint is reclaimed with a high wall, navigation of the channel through the gap appears to be rendered quite easy around high water.

Scheme E greatly prolongs the time from the beginning of the flood to high water at Connah's Quay, while having the effect of reducing flood-velocities and increasing ebb-velocities in that region. Between Connah's Quay and Chester weir, high-water and low-water levels with the river in flood are lowered appreciably.

As the level of high-water springs is approximately +29·00 L.B.D. between the mouth of the walls and Connah's Quay, it follows that if dredging were carried out between the walls to maintain a level of, say, +9·00 L.B.D. near Connah's Quay, the depth at high-water springs would be 20 feet near Connah's Quay and 30 feet at the mouth of the walls. The depth at low water spring tides would be approximately 3 feet throughout.

(*f*) The possibilities of a barrage near Connah's Quay have also been studied, but the results obtained do not justify the conclusion that such a barrage would be effective. On the contrary, they indicate that, except in the immediate vicinity of the sluice-gates, the seaward channel would deteriorate.

Acknowledgements.

The Author desires to express his sense of indebtedness to Professor A. H. Gibson, D.Sc., LL.D., M. Inst. C.E., Director of the Whitworth Engineering Laboratory of Manchester University, and to Captain G. A. Wright, M.C., Chief Engineer to the River Dee Catchment Board, for whom the investigation was undertaken; also to the Author's colleague, Mr. J. L. Matheson, M.Sc., Assoc. M. Inst. C.E., and to Mr. C. H. Ku, B.Sc., research student, who have collaborated in both the experimental work and the plotting of the many charts, only a few of which are reproduced in this Paper.

The Paper is accompanied by nine sheets of drawings, from which Plate 2 and the Figures in the text have been prepared, and by three photographs.

Discussion.

Mr. Elsden showed a number of lantern-slides illustrating the work described in his Paper.

Mr. Allen showed a series of lantern-slides in illustration of his Paper.

Sir Leopold Savile, Vice-President, wished to express his regret that his senior partner, Sir Alexander Gibb, Past-President Inst. C.E., had not been able to be present that evening, because Sir Alexander was particularly interested in the model and had really been responsible for recommending to the Rangoon Commissioners that they should deal with their problem by means of a model. It would be generally admitted that Sir Alexander had taken a bold step in embarking on a model-experiment involving the expenditure of about £10,000 at a time when the advantages to be obtained by such an experiment were considered by many engineers to be very doubtful. That Sir Alexander was justified in his decision would be clear, as in spite of the fact that the navigable depth of the channel had been growing less for many years, it had been possible to advise the Commissioners, as the result of the experiments, that the deterioration would appear to have reached its limit, and that there was a reasonable prospect, without any large expenditure being undertaken on walls or dredging, of the conditions improving, or at any rate not deteriorating any further. That that advice had been justified so far would be borne out by Mr. J. Guthrie Brown, M. Inst. C.E., later in the discussion.

It would no doubt be realized that the results of the model-experiments could not have been obtained but for the work done on models by Professor A. H. Gibson, M. Inst. C.E., and others. It was due to the work that Professor Gibson had done and to the generous help that he gave that it had been possible to take advantage of the practical problems which he had already solved and to develop the model in the way that had been done. In addition to the experience gained in former work on models, reference should also be made to the enormous amount of work which had been entailed on the part of the technical officers of the Rangoon Port Trust, because it would be readily realized that a large amount of data and a great many observations had to be obtained and made; Mr. W. D. Beatty, M. Inst. C.E., the Chief Engineer of the Rangoon Port Trust, and his engineering staff, and Commander C. M. L. Scott, R.N. (retired), Deputy Conservator, and his surveying staff, were to be congratulated on the manner in which they supplied all the requirements under what were very often exceedingly difficult and hazardous conditions.

Last, but not least, Sir Leopold wished to record the great appreciation of himself and his partners of the fact that the Governing Committee of

University College, London, had placed a large basement at the disposal of the Rangoon Port Commissioners, and that Professor E. G. Coker, M. Inst. C.E., and later Professor G. T. R. Hill, of the Faculty of Engineering, and Professor F. G. Donnan and Mr. Henry Terrey, of the Faculty of Chemistry, had rendered most valuable assistance. It was only right to record that the satisfactory results obtained would not have been possible without the assistance and advice which had been received.

Mr. G. J. Griffiths remarked that he would confine his remarks to Mr. Allen's Paper. He had always considered that sooner or later there would be a controlled channel extending downstream some considerable distance below Connah's Quay, and he was very glad to see that the matter had been so thoroughly investigated. At the same time he would like to utter a warning in regard to the results of the model-tests, particularly from the point of view of the erection or forming of training walls. The sands in the Dee estuary were very unstable, and great care would be necessary to ensure the stability of the foundations of the training walls. It might also be possible to construct groynes at right-angles to the training walls, and hence gradually to reclaim the whole of the foreshore.

The Paper showed how valuable the use of aerial photography was in surveying an estuary of the kind in question, and in fact for all classes of work. He had not the time to consider all the details of the various schemes which Mr. Allen had investigated, but from a wide experience of both the Mersey and the Dee estuaries, he felt sure that those concerned were correct in not trying to keep the controlled channel too near to the Flintshire coast; he thought that it might be well that they should carry out additional investigations with regard to the width of the mouth of the controlled channel before they proceeded further into the matter, because that factor was all-important.

He agreed that a barrage would be useless. With regard to the training walls, he hoped that the southern wall would always be kept above the level of high-water spring tide, its extensions being gradually followed by those of the northern wall, also kept above spring-tide level, so as to assist in the reclamation of both foreshores.

Captain G. A. Wright said that he was the instigator of the investigation of which Mr. Allen had given such an interesting account. The lack of financial resources had prevented the River Dee Catchment Board from carrying out an instrumental survey of the Dee estuary and it could, therefore, be said that the Board's poverty had resulted in the study by Mr. Matheson of plan-projection from obliquely-taken aerial photographs, and subsequently in the construction of the model described by Mr. Allen from the survey prepared in that way. He wished also to point out that, although the river was called the Cheshire Dee, only 5 miles of its course lay in Cheshire, the remainder being in Wales or forming the boundary between Wales and England.

Mr. Oscar Borer said that he was particularly interested in the size

of the bed-material chosen for both the models described in the Papers. In the case of the Rangoon model, he noticed that Mr. Elsden stated that the grain-size was approximately three-quarters of the size of the material found in nature, a proportion which Mr. Borer had found to be fairly satisfactory. What size sand had Mr. Allen used in his models?

In the Rangoon model silt was introduced in suspension. Since the banks were referred to as being alluvial, presumably some form of alluvial silt was introduced. In Mr. Borer's experiments alluvial silt had been introduced, but had produced a very bad effect; when that came down with the fresh water and entered the salt-water portion, representing the sea, there was at once coagulation of the material and it settled on the bed, so that after the first few tides—a matter of a few months in natural time—the whole of the bed coagulated. He would be interested to know whether any effect of that kind had been found in connexion with the Rangoon model.

Mr. Allen stated that no silt in suspension was supplied to the model of the river Dee. He assumed that the Dee carried silt only at certain times, and that therefore the investigations had been confined to the movement of the bed itself. It sometimes seemed to him that discussions in regard to silt in suspension did not lead very far, because, after all, if material were in suspension it did not matter whether or not it moved up and down the river, and he thought that Mr. Allen had discussed the problem correctly; it was with the movement of the grains of sand along the bed that the engineer was concerned.

Mr. Allen had used a second model, and Mr. Borer thought that that had gradually come to be the practice with regard to tidal models. At the Poona research station three models had recently been used when dealing with tidal problems. It was not possible to be quite sure that one model was going to give the exact results, and since there was a great sum of money involved in the actual works, it was desirable to check the results by means of a second model.

On p. 38 Mr. Allen referred to a strong ebb current impinging on the foreshore seaward of the walls, the current being apparently induced by the construction of the walls themselves. That was an effect which might be expected, because if an attempt were made to divide an estuary artificially by putting in high training walls, there was sure to be some portion of the estuary which would not receive its proper amount of tidal water, and therefore a cross current would be bound to be set up in the endeavour of the tidal waters to restore equilibrium throughout the whole estuary; the inevitable result of that, especially with high training walls, would be to set up a flow behind the walls. He knew that the experiments with regard to the Dee walls had not continued for a very long period of time, but from what he could see of some of the soundings that were given, there seemed to be a tendency for a scour to be set up at the back of the walls. If that were so, it would make the construction of the walls extremely difficult. Such a tendency was natural, moreover, since, as

the water was coming off from the estuary and could no longer find its way into the channel into which it naturally flowed, it naturally flowed against the only obstruction which could possibly bear against it, and so gradually the water from either side of the training wall would tend to flow outside it. If the channel were partially filled with silt, however, there would probably not be much tendency to scour it clear again. The construction of the training walls would in any case not be easy.

He wished to make a brief reference to the use of the barrage. Mr. Allen referred to the fact that a hole appeared below the barrage, apparently suggesting that the fresh water was held up, and said that it was discharged in the lower part of the ebb. If the gates were only partly opened, so that there was a discharge underneath them, with an artificial head, local scour was certain to occur, with a great deal of shock through the head of water being discharged into the area below. If, however, as soon as the level at which it was desired to operate the barrage (perhaps 1 or 2 feet below the impounded-river level) was obtained, the gates were opened fully, local scour was no longer created; the surface-slope of the whole river would be steepened, and the effect would therefore be very different from that of leaving the gates partially closed and thereby inducing local scour. A great deal of investigation would be involved if there were to be any hope of a barrage being at all successful on the river, and he would be interested to know whether or not the Author had attempted to see what would happen if he simply made use of the water stored behind the barrage to create a greater head, and therefore a greater surface-slope, which would have an effect for some considerable distance below the barrage itself.

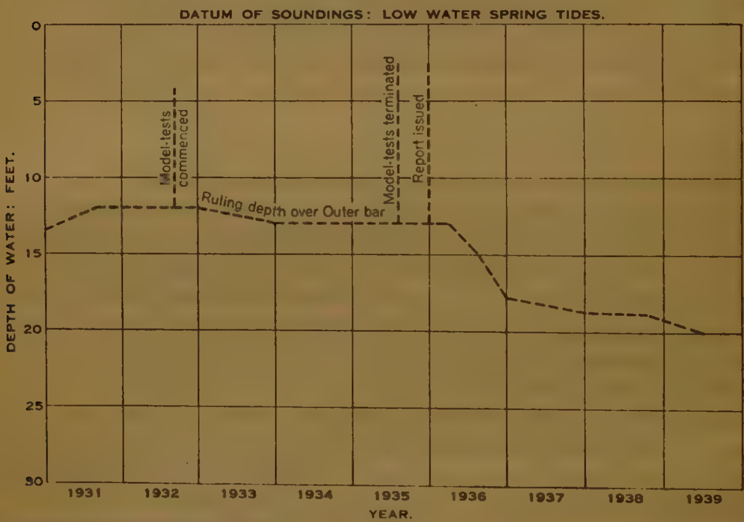
Mr. J. Guthrie Brown observed that the Rangoon model was based on the principles brought into the realm of practical engineering by Professor A. H. Gibson, M. Inst. C.E., in recent years. It was doubtful whether a model had ever been used before to deal with such a large number of variable factors, and with so little information at the start with which to allow for them; at the time it was the largest tidal model to be constructed in Great Britain.

The behaviour of most models was extremely temperamental, especially in their early stages, and there were many initial difficulties to be overcome in the Rangoon model. The extensive problems which had had to be solved were very well described in the Paper, and satisfactory information had been obtained on all the unknown factors. The confidence felt in the accuracy of the model in foretelling the future enabled Sir Alexander Gibb to indicate, as a result of its operations, that the conditions at the bar, which had suffered as the result of deltaic degeneration of the river mouth, with consistent loss of navigable depth, for the greater part of a century, would appear to have reached their climax. The Commissioners, who had contemplated as an essential requirement an extensive and expensive system of river-training, were advised that any such expenditure would be worse than useless, that a policy of masterly inactivity was all that was

necessary, and that the conditions would become no worse and would in all probability improve. Such advice would have been inconceivable without the guidance of the model, and it was gladly accepted by the Commissioners. Whilst that was most gratifying to those concerned with the model, what was even more encouraging was that the conditions had improved as forecast by the model.

The first reasonably detailed chart available of the river mouth was dated 1860, when there was a ruling depth over the bar of 24 feet at low water spring tides. This depth continually decreased until in 1931 the depth available was only 12 feet; the model-experiments were then commenced. *Fig. 1* showed the depth of water over the bar during the last 9 years. There had been a period of 3 years, during which the model was

Fig. 1.



RANGOON OUTER BAR: RULING DEPTHS AT LOW WATER SPRING TIDES.

operated, when the 12-foot ruling depth was still about the same. Up to that time there had been no justification for hoping for an improvement. The model-results, however, indicated that the conditions would, in fact, improve, and in due course they had started to improve; the results had proved the accuracy of the forecast, because up to date there was a ruling depth of 20 feet over the bar, there having been a gradual and continuous improvement.

Mr. J. M. B. Stuart said that on p. 7 of Mr. Elsden's Paper were given some of the possible causes of the trouble at the outer bar. No. (3), "Silt brought down by the other mouths of the Irrawaddy and washed eastwards by tidal streams", and No. (5), "Changes in the tidal streams due to widening, straightening, and other changes in the Rangoon river mouth", were probably the most important factors; No. (3) was probably

a more or less constant factor over a number of years, whereas the changing factor was No. (5).

The purpose of the model was to find the cause of the growth of the outer bar and to ascertain whether or not anything could be done to improve conditions there. There were apparently no questions concerned with the Rangoon river itself to be investigated by the model, and, whilst there might have been good reasons for what had been done, it seemed to him that it might have been possible to have dispensed with the labyrinths of the Panhlaing and Pegu rivers and of Pazundaung creek, and to have had one combined labyrinth for the Rangoon river and the channels which ran into it at the upper end of the Hastings shoal. That would have simplified the model-arrangements.

On p. 17 it was stated that provision was made in the case of the Sittang labyrinth to reproduce the effects of the Alok cut, which was a cut-off which shortened the course of the river by between 30 and 40 miles. On p. 13 the Author seemed to have gone away from the idea of representing things accurately as they happened in the Sittang river, because it was there stated that only one-half of the calculated discharge was supplied to the model, it being considered that only half the actual river could be said to flow into the modelled area. He would like to know why those comparatively small channels, such as the Twante canal and the Bassein creek, were included in the model; they could hardly have any effects on the problem under investigation.

The mouth of the Sittang river oscillated from side to side over periods of time. During recent years there had been an accretion on the east side of the Gulf of Martaban, and erosion on the western side of the mouth of the river. It was possible that those changes in the gulf and the coast-line might have had some effect on the increase in height of the Outer bar. Great trouble had been taken in the model to give special consideration to all the rivers that entered the area, but it seemed to him that the coastline and the changes in the gulf could have been considered only very vaguely in the model.

On p. 28 the final decisions were mentioned. The Rangoon Port Commissioners apparently accepted the advice of Sir Alexander Gibb and Partners and did not carry out any dredging, and they were wise in so doing, but it was rather doubtful whether there was any necessity for them to announce that in future Rangoon would be only a second-class port, because from what Mr. Guthrie Brown had said it would appear that the conditions were now much better there than they had been. It was unwise for the Commissioners to say that they could not take any vessels drawing more than 28 feet of water. At that time—1935—Rangoon was only a port for Burma, but in view of the war in China it was now a port for western China also, and might be of greater importance in the future than it had been in the past.

Mr. John McClure observed that it would be of interest if Mr. Elsden could explain what provision had been made in the tidal mechanism for

producing the natural period of rest, varying from, say, 20 to 30 minutes at high and low water, as such periods of rest had an important effect on the deposition of the silt. The peaks of the model tidal waves in the diagrams shown in Figs. 8, Plate 1, seemed somewhat sharp, and would indicate a lack of still-water periods.

After 3 years of model-investigation, the final decision reached had been to permit present conditions to remain and to limit the future draft of vessels using the port, which seemed somewhat disappointing; in that connexion it would be of interest to know whether any model-experiment had been made with the proposed dredged channel shown in Fig. 9, Plate 1, using half-tide banks to form a funnel entrance to the cut. Clay seemed to be available for the purpose, and the advantage of using clay was that if, shortly after the commencement of the cut, it were found that no good purpose was being served, it would be quite easy to dredge it up again, whilst if the effect of the channel were really satisfactory it could be made more permanent. In a similar case of a long dredge and curved entrance-channel with which he had been concerned, such a hard clay funnel-mouth bank had been formed, with very good results in the maintenance of the channel, and afterwards it had been improved with rubble on the channel side to render it more permanent.

Referring to Mr. Allen's Paper, he wondered whether any experiments under scheme E had been made, especially on ebb tides, by actually forming a channel. Had any experiment been made by drawing the sand aside and inducing the channel to avail itself of the ebb flow which was referred to? If in the model any advantage had been found from that experiment, it seemed to him that it would not be very difficult, with a big sand-dredger depositing sand from that channel (not the full depth at once), to introduce it, with the proviso that the sand which was dredged from the cut had to be introduced under water-level at the banks, and not above water-level. If that were done, it would be found that the sand would stay there, and would probably not be readily carried away by the currents.

Mr. J. L. Matheson remarked that the great complexity of the regime in the Rangoon estuary and the various points which had been taken into consideration, such as silt and coast-erosion, contrasted with the comparative simplicity of the Dee estuary, which was almost free of silt, so that it was not necessary to introduce it into the model.

That contrast between the two models was reflected in the tide-curves. The Rangoon tide-curves showed a slight divergence between those of the model and those actually obtained in the estuary, no doubt due to the great variety of influences which affected the Rangoon tide. On the other hand, the tide-curves of the Dee showed an almost exact agreement between the tide in the model and the tide in the estuary.

Turning to the question of the Dee, he showed two lantern-slides which illustrated the regime in that river. A survey of the Dee in 1732, which had been made by the opponents of the scheme of training walls, showed

that the deep-water channel clung to the north shore of the estuary. The training walls had been constructed about 1750 and deflected the channel to the south shore, but seaward of the walls the channel reverted to the north shore near Parkgate (*Figs. 1*, p. 31), which at that time was a port of some importance. That tendency of the river was shown exactly by the model itself, and it was interesting that every attempt which was made to restrain it on the south side met with considerable opposition from the river itself, which tended to revert to its old course along the north bank. It was also particularly interesting to notice how the S-bend in the river near Parkgate, which had been shown on some of Mr. Allen's lantern-slides, was developed in the model with remarkable accuracy.

Mr. Ll. E. Williams, Assoc. M. Inst. C.E., had found the speed of the bore in the river between Connah's Quay and Chester to be 8 miles per hour. The speed in the model, when converted to full-scale units, was 7·94 miles per hour.

Mr. G. A. Maunsell had taken part in a preliminary investigation into the problem of the Rangoon Outer bar. That investigation had comprised both a study of old records and an examination of the actual conditions near Rangoon, where he spent several weeks in 1929. As a result of that investigation the conclusion had been reached that training banks offered very little prospect of success within reasonable limits of cost. He was, therefore, very interested to see that the third stage of the model-experiments described by Mr. Elsden—namely, the attempts to see the effect of training banks in a model—showed that the conclusions reached from the preliminary investigations were sound.

Mr. Elsden also mentioned the very interesting fact that the mouth of the old Rangoon river might possibly have followed quite a different course about 200 years ago. That was the conclusion that had been reached as a result of the examination of old records. Such a change in course was not inherently unlikely in that neighbourhood, because the whole district was a vast natural laboratory where great changes in the river regime were going on at an extremely rapid rate; for example, in the Sittang river, a little higher up the Gulf of Martaban, there was a pronounced bore, and within living memory a big cut-off had occurred whereby a 40-mile length of fairly wide estuary had been eliminated.

Mr. Elsden found that it was impossible to find a material which would produce the effect of erosion in the model, and he described how, at intervals during the experiment, he used to pare away by hand banks which were eroding in practice, whereas the model by itself was capable automatically of producing the silting effect by accretion. Mr. Allen, on the other hand, was able to produce the effect of erosion as well as of accretion in his model, and it had been mentioned that the model of the Dee was a very much simpler problem on account of its being just a matter of a fresh-water discharge combined with and the flux and reflux of tides over a large sandy estuary, whereas in the Rangoon model there were two different kinds of accretion going on, the accretion of sandbanks rolling

down the river-bed and accumulating just at the mouth of the river, and the accretion due to siltation. To what extent in the reproduction of the natural accretion in the Rangoon model was it possible to discriminate between the accretion due to sandbank formation and the accretion due to silting? Was it possible in the model to produce both sandbanks and mudbanks? The bar itself, which extended about 6 or 7 miles out to sea, consisted of a huge flat expanse of very soft mud. Just outside the mouth of the Rangoon river, where the river channel was itself very deep (that was to say, nearer inshore than the great mud bar mentioned above), the river had piled up enormous banks of sand on either side of its channels. Those sandbanks were of a firm compact description, and great expanses of them were exposed at low tide. The bottom of the river-channel in the Rangoon river itself were almost everywhere composed of sand or gravel. There were deposits of fine silt in backwaters of the channel and the banks of the channel were mudbanks which had been cut through by the river and which were being eroded in many places. The bottoms of the active channels were, however, almost invariably covered with sand. Mr. Maunsell therefore came to the conclusion that there were two totally independent sources of accretion, the one being sand which was rolling or drifting down the bed of the river and which was ultimately finding a resting place in the sandbanks formed near or just outside the mouth of the river, and the other being a very fine argillaceous silt which was being deposited everywhere over the whole bottom of the Gulf of Martaban wherever the movement of the current was sufficiently sluggish to permit deposition to take place. For many miles off shore the water in the Gulf of Martaban had the appearance of very thin pea soup, caused by the presence in the water of that impalpable silt, brought down partly by the Rangoon and Sittang rivers but mainly by the great rivers draining from the central Asian mountain plateau, namely the Irrawaddy and the Salween.

The problem of the Outer bar at Rangoon was quite evidently a case of silt-deposit from the silt-laden waters of the gulf, and he assumed that Mr. Elsden's model-experiments were mainly directed to reproducing that silt-deposit. However, the fact should not be lost sight of that the other deposit of sandbanks, just inside and just outside the mouth of the river itself, was quite a separate phenomenon which was going on all the time, and which probably exercised a determining influence upon the direction of the channels at the mouth of the river and to a large extent regulated the erosion in the banks of the river at Elephant Point and elsewhere. Had an attempt been made in the model to reproduce that sand phenomena simultaneously with the reproduction of the silting phenomena upon the Outer bar?

Mr. Maunsell wondered if Mr. Elsden wished to qualify his statement that the model-experiments were based upon empirical formulas originally devised by Professor Osborne Reynolds. It appeared that the use of the word "empirical" might give the impression that there was no basis in mathematical first principles for model-experiments.

Did Mr. Elsden consider that it would be possible to make model-experiments reversible? That was to say, starting with present-day conditions in the tidal estuary, would it be possible to devise tidal movements which would in effect run backwards in time, and which would reproduce conditions of erosion and accretion in the reverse order so as to finish up with the state of the estuary such as it did in fact occupy in time past?

Mr. W. E. Doran said that both the Papers gave very interesting accounts of model-experiments, but that as time was limited he proposed to confine his remarks to Mr. Allen's Paper on the Dee. It was of interest to note that when the North wall was extended, the scour only extended a short distance beyond the toe of the wall, and that material was in fact carried upstream rather than the reverse. Mr. Allen's statement with regard to the same result being obtained with three different bed-materials related to what appeared to be a fundamental point in investigations of that kind, namely, the selection of the bed-material. The Author had not mentioned that any special investigations were made in that connexion, but no doubt such an investigation had been carried out. It had been found in model-experiments generally that the rate of scour depended chiefly on the grain-size and the specific gravity of the material, and that the model time-scale could not therefore be applied to the rate of movement of bed-material. From results which had been obtained with models in the past it seemed that in many cases, even when special precautions had not been taken in the selection of bed-material, the models did reproduce their prototypes quite closely, and he felt somewhat puzzled why that should be so, because it appeared to him that unless the rate of movement of the bed per tide in the model corresponded to the rate of movement per tide in nature, a cumulative error was bound to take place, which, in a relatively short time, would change the channels, and hence alter the direction of the currents and therefore the tide-curve.

Obviously the rate of scour for powdered pumice and for somewhat coarse sand would be quite different, and again the critical velocity at which movement began and ceased would also be different; apparently, however, the results in the same number of tides were very much the same, and no cumulative error appeared to take place. He would be very glad if Mr. Allen could explain that apparent contradiction.

He felt that the question of bed-material was of very great importance, because unless its behaviour could be relied upon the validity of the conclusions as a whole would have to be questioned. In the present case it apparently had not been possible to prove the fidelity of reproduction of the model by selecting a known state in nature, or, rather, two known states separated by a time-interval, and then running the model in a corresponding manner, in order to see whether the second state in the model corresponded to the second state in nature. In the absence of such evidence of the accuracy of the model-predictions, the results depended essentially on the question of bed-material movement.

He noticed that the model showed a satisfactory stability over 9·64 years after the conclusion of the works, and he presumed that although the depth of water at low water spring tides was only 3 feet at the end of the training walls, the stability of the channel could be relied upon. Was there reasonable certainty that storms and other factors which could not be provided for in the model would not cause the channel to alter, and thus perhaps invalidate the effect of the training walls? In other words, did the model reproduce all the really important factors which governed the changing of the channels?

With regard to the experiment on the barrage, if the barrage were shut it cut off the very important ebb flow of the tide, which would be bound eventually to cause siltation of the estuary, and he wondered whether any experiments had been tried of keeping the sluices open up to the maximum flood tide and then closing them for an interval. In that way the ebb flow of the portion of the river above the sluices would still take place and the necessary increased velocity would have been obtained in the ebb flow; that increased velocity might possibly have prevented siltation below the sluices.

In conclusion, he thought that Mr. Allen's Paper illustrated in a most admirable manner the very great value of model-investigations in regard to a tidal estuary. It was only necessary to consider the five projects shown in *Figs.* 7 (pp. 42, 43), and numerous others which were bound to have been tried. Without the help of a model it would have been impossible to have assessed with any degree of accuracy the precise effect of each project. Again, in the case of the barrage it had been thought that a channel 6 feet below ordnance datum could have been scoured out, but the model was able to show that such would not have been the case. In view of the directness of the results and the ease with which variations could be tested in a model, he thought that no engineer could be justified in failing to test his theories in that way before putting them into practice.

*** **Mr. M. F.-G. Wilson**, Vice-President, thought that everyone would be impressed by the great care and forethought with which every factor affecting the question had been considered, and every imaginable contingency provided for, when working out the details of the Rangoon model, more so perhaps than was often the case. Local conditions were also very difficult to provide for, the difference in the flow between the dry-season flow and times of heavy floods being practically in the proportion of 1 to 25. A first impression might be that the model had been worked out in almost too elaborate detail, and that time and cost could have been saved by adopting a less complicated apparatus, but closer consideration corrected that impression. It was, he thought, often difficult to appreciate the great proportional difference in the real size of a model of the kind in question as compared with nature, as the idea of size was governed by the horizontal scale of so many feet to the inch. In the present case if the

*** This contribution was submitted in writing.—SEC. INST. C.E.

three scales were multiplied together it would be found that the volume of the model was only about one twelve-thousand-millionth of nature. Where such great disproportions were involved it was obvious that too much care could not be taken in regard to any detail affecting the construction and working of the model, even if that should involve considerable expense, for without such care the valuable and reliable results so often obtained from models, and which quickly repaid their cost, could not possibly be attained. That as a result of the experiments the Rangoon Port Commissioners were enabled to come to the important conclusions referred to on p. 28 was ample proof of the value of the experiments and the reliability that could be attached to them.

The accuracy with which the actual tides were reproduced was clearly indicated by the close similarity between the curves of the natural and model-waves, as shown in Figs. 8, Plate 1; in the case of the Severn Barrage model there was an even more striking check on the accuracy of models: floats put into the river near Avonmouth about the time of low water were carried up by the flood, some of them grounding on the river bank when the tide turned. Similar floats put into the model were carried up to, and grounded at, almost the corresponding places.

Nevertheless it seemed to Mr. Wilson that, in spite of the good results obtainable from a carefully made model, they should always, at any rate in certain directions, be treated with caution. The general indications might no doubt be accepted with very considerable confidence; for example, if the model showed that in certain places or over certain areas deposition and shoaling occurred it might be assumed with great confidence that changes of a similar character would take place in nature. Further, the indications given by the model regarding whether the observed changes would be slow or rapid might no doubt be relied upon. The extent of those changes, as affecting the actual depth of the channel at any given spot or over any particular area, should not, however, be accepted with certainty. He had on several occasions noted a marked difference between the precise indications of the model in that respect and the actual results in nature. Those discrepancies might perhaps be accounted for, at any rate to some extent, by the great difference in the proportionate size of the sand-particles in the model as compared with nature.

It was stated that the wind fans were regulated to produce waves of the correct height from crest to trough, "a factor considered to be of more importance in this case than length or speed." It would be of interest if the particular reason for that opinion in connexion with Rangoon were given. At Colombo the height of the long heavy south-west monsoon waves seldom exceeded about 15 feet, yet they struck the breakwater with such force that on one occasion while the works were still under construction the whole outer end of the structure, consisting of 32-ton concrete blocks, was slewed bodily inwards due to the weight of the wave-stroke.

The Dee model was built to a considerably larger scale than the Rangoon

model; that could be easily and advantageously done because of the much smaller area to be dealt with. What was particularly interesting to note in the Dee model was the difference in the direction of the main conclusions drawn from it as compared with those drawn from the model of the adjoining river Mersey, although the physical conditions obtaining in the two cases were naturally very different. There was a large reservoir of tidal water in the Upper Mersey, the filling and emptying of which was shown by the Mersey model to produce sufficient scour to render high training walls unnecessary; in fact a trial of such walls in the model proved them to be completely unsatisfactory, and even harmful. There was practically no upper reservoir in the Dee, however, and the experiments with the model showed that training walls, reaching, over a considerable proportion of their length, to high-water level, were necessary.

Mr. Elsden, in reply, pointed out that the materials used to represent alluvial silt in the Rangoon model were commercial clays, chosen after exhaustive tests to determine whether or not they would settle through water at the correct scale rate.

The coagulation-effect mentioned by Mr. Borer had been definitely obtained in the Rangoon model. In that respect, the model was behaving in a manner entirely true to real conditions: the coagulation-effect was, in real estuaries, an important factor in producing silty deposits where fresh-water rivers entered the salty waters of the sea, and it was essential that the model should reproduce that phenomenon.

Mr. Elsden could say definitely that the model had reproduced both sandbanks and mudbanks in the mouth of the river, and in answer to Mr. Maunsell's question, he would say that the silt- and sandbanks could readily be distinguished when the model was being surveyed. As mentioned on p. 22, the travel of the silt was traced by means of a series of chemical tests. That process entailed the taking of a large number of bed-samples at the close of each run of the model; those samples showed at once whether a shoal was formed of sand or of silt.

He considered that coastal erosion was far more difficult to reproduce in a model than was the local erosion which was bound to accompany the movements of the various shoals in the bed of an estuary. The latter type of erosion had appeared in the model, but Mr. Elsden had found extreme difficulty in reproducing the coastal erosion between Bassein creek and Elephant Point; that was to say, that along the west bank of the river. In that case the natural side slope was very steep, and became sensibly vertical when modelled to the exaggerated vertical scale adopted. To save time in further preliminary experimental work, Mr. Elsden adopted the procedure explained on p. 21. When that had been done, the sandbanks to which Mr. Maunsell referred developed in the model in a manner similar to that followed by the actual shoals.

Mr. Elsden did not intend to deny the existence of a mathematical basis for hydraulic models, but tidal models were, he understood, usually proportioned on an experimentally-determined relation (due to Osborne

Reynolds) between the range of the tide in question and the scale exaggeration of the model.

He did not consider that the development of an estuary was a reversible process, though it might easily be cyclical, and the regime might revert to an earlier phase. A "reversed" model could, of course, be constructed, but in Mr. Elsden's opinion it would merely show what would happen if the direction of flow of the rivers and tidal streams were reversed. He did not consider that models could be used for historical research.

No special means had been adopted to produce slack-water periods in the model tides. The curves given in Figs. 8, Plate 1, showed that the model-tides were rather flatter than the actual curves at high-water spring tides; the reverse was the case at neap tides.

He agreed that a closer agreement could have been obtained between model- and actual tides by prolonging the experimental adjustment of the tide-engine. The number of actual tide-curves obtained from Rangoon had, however, to be severely limited owing to the difficulty of taking long series of readings at such stations as Elephant Point and China Bakir. As the actual tides were subject to considerable meteorological variations, it was not considered that further refinements in the model-tides, to make them agree in detail with a particular set of actual curves, would be warranted, particularly as the model-results were only to be accepted after they had been exhaustively proved over a long period of past history.

In answer to Mr. McClure's second question, no scheme of half-tide walls had been tried in connexion with the dredged channels. The actual material dredged would not be suitable for the construction of such walls, and they would not otherwise be an economic possibility.

Mr. Elsden agreed with Mr. Stuart that, as things turned out, the scope of the model could have been limited to the region below the Pegu entrance. When planning the model, however, it had been considered best to include the whole of the inner harbour, in case it should be desired to investigate further problems in that area. The coast-lines in the upper part of the Gulf of Martaban were only very vaguely known in actual fact, and the earlier charts from which the experiments had been started were very vague indeed. Had more definite data been available, those coast-lines would have been reproduced in greater detail.

Mr. Elsden agreed with Mr. Wilson's view that model-results should be accepted at any rate with some caution. He felt that it was not yet possible to make accurate predictions of channel-depths.

The reason for adopting height as the criterion for model-waves was that, in a model constructed to the scales adopted in the Rangoon experiments, it was the only practical criterion to adopt. Waves of the correct length, even if considerations of surface-tension allowed them to be modelled, would be so small as to have no effect on the bed-materials. Waves of the correct speed, or with correct energy, would be impracticable for similar reasons.

Mr. Allen, in reply, said that, in regard to the width of the channel between the training walls, various widths had in fact been investigated; the results obtained with the model had been such as to indicate that the widths given in the Paper would be the most successful.

With regard to the barrage, as tried in the model no tide water passed the sluices on the flood tide above the site of the barrage, but river water which accumulated above the site of the barrage during the time that the gates were closed was discharged on the ebb. The possible variation of admitting a certain amount of flood water through the sluices and using that for scour on the ebb tide had not actually been tried, but he did not think, from the general run of the experiments, that there was much possibility of obtaining anything but a local improvement by means of any possible arrangement of barrage.

The bed-material had been mentioned. A sand the diameter of which was about 0·007 inch had been used, and the ratio of that to the sand of the natural estuary of the Dee would be about 3 : 4, or about the same as in the case of the Rangoon model; certain vital tests had, however, been tried with other bed-materials—a coarser sand, of about 0·009 inch diameter, and a powdered pumice, much coarser in size but much lighter—and the results had been sensibly the same. Two very different model-scales had also been tried, as detailed in the Paper, and the results obtained had been sensibly in agreement on all issues. The time taken for the model-investigation of the Dee was about 15 months.

The practical consideration of the construction of any such scheme of walls in the actual estuary had been referred to, but that was a problem which was outside the scope of the Paper. The object of the investigation, in his view, was to study scientifically (or so, at least, he hoped) the hydraulic advantages of various possible schemes. Constructional difficulties, as apart from hydraulic effects, had not been dealt with in the Paper, but it might be mentioned that the opinion of river-training engineers had been sought on that question, and the opinion had been expressed that it would be practicable to construct such a scheme of walls, using dredged sand for the core of the walls, with a pitching of stones and the principle of the mattress.

It was especially gratifying to have the opinion of Mr. Griffiths that it would be incorrect to try to keep the controlled channel too near to the Flintshire coast. Regarding the possible programmes of extending the walls, it might be of interest to state that since the Paper was written an alternative suggestion had been investigated; namely, the construction of the walls of scheme E by commencing at the seaward end with the North wall, and bringing that gradually upstream for some distance before carrying out operations on the raising and extending of the existing walls. Further, in the final arrangement the South wall was not continued downstream beyond section 11, thus saving approximately 2 miles of wall as compared with the scheme as shown in Fig. 8, Plate 2. On the whole, that arrangement had not proved as satisfactory as the

original scheme E: there had been a movement of material from the Bagillt bank into the channel seaward of section 11, and consequently an increased volume of dredging had been required to maintain as good a channel. Moreover, the beginning of the flood tide at Connah's Quay had proved to be quicker, and the latter part of the ebb slower, than if the South wall, as well as the North wall, were extended to the neighbourhood of section 15.

Whilst it had been pointed out that the intrinsic constructional problems involved in such works were not within the scope of the model-investigation, it was appropriate to point out that a model might be of some assistance in that respect: for example, the model of the Dee indicated that extra protection of the back of the North wall would be desirable in the region between sections 10 and 11, to withstand the impact of the flood current proceeding along the old channel. An effect of that kind, as also any cross currents likely to cause difficulty either in the construction of the walls or in navigation between them, was demonstrated very clearly in such a model.

Concerning Mr. McClure's query, the model did not lend support to the conclusion that a dredged channel would be adequately maintained without training walls.

Mr. Wilson's reference to the different conclusions that were naturally derived from the model of the Dee and one previously constructed at Manchester of the Mersey, in respect to the desirable height of the training walls, was certainly significant in support of the contention that every river presented peculiar features, and, if major works were contemplated, deserved its own model-investigation. It was equally true, as Mr. Wilson observed, that model-results were to be interpreted with caution, and Mr. Allen was of opinion that it was important to carry out pairs of tests, starting from the same initial condition, with and without proposed works, in order to obtain a direct comparison.

Mr. Borer and Mr. Doran had raised the important question of bed-materials. It was true that in the Dee investigation no test had been carried out to compare the behaviour of the models over a period of time corresponding to two surveys in the natural estuary. On the other hand, the method of comparative tests, with and without proposed schemes, had been used, and two models of very different scales had been employed as an indication of possible "scale-effect."

The general subject of "calibration tests" from the state of an old chart to present-day conditions deserved consideration. Certainly in more than one investigation, the reliability of a model had been thoroughly proved by such tests, but there was no rational cause for anticipating that, having moulded the bed to some previous state—say 50 years past—and running the tides up to date, the model would show, in any given case, the present configuration of the estuary, unless the information was available, and the mechanical devices were practicable, to reproduce gales and other such "spasmodic" occurrences as might give rise to some

alterations in nature. A model would only show the general effect of normal tidal action, although a reasonable attempt might often be made to reproduce abnormally high or low tides, or flood or drought conditions of the rivers, if the necessary data for the estuary itself were provided.

If, as in the upper estuary of the Severn, the currents themselves were very strong, it was proved by experiment that close agreement with nature over a long period of time was obtained. In general, however, a model might be expected to show the probable results of proposed works in conjunction with normal tidal action, and a qualitative estimate of gale-effects might be made by visual observation of the model, which might indicate, for instance, that gales from a certain probable direction would carry bed-material over the walls and into the trained channel of some suggested scheme.

The question of rate of movement of the bed had to be considered in relation to the duration of the experiments. In non-tidal work on the scour at the toes of dams, it had been found by Messrs. R. V. Burns and C. M. White * that, whilst the rate of scour with various materials was quite different, the final condition reached was almost identical (within limits of grain-size). Accordingly, if the test were run sufficiently long for the establishment of approximate stability, similar conclusions would be reached. In tidal work, however, the alternating effect of flood and ebb greatly assisted in making the rate of change of the bed approximately the same for materials of different kinds (within limits). Thus, in an extreme case, if a certain material were moved equally up and down stream by flood and ebb respectively, the final result would be identical with that obtained if the bed-material had not been moved at all.

Mr. Allen would invite the attention of those interested in the theoretical and experimental subject of the effect of grain-size, shape, and density, to Professor A. H. Gibson's Severn Model Reports, Sections IV and V and Appendix D †, and to pp. 708–713 of his Vernon-Harcourt Lecture ‡; and again, concerning the matter of silt in suspension and its disposition in a model, to his Severn Reports, Section XIV *.

In conclusion, he would like to endorse the view that both Mr. Elsden and himself had been very fortunate in being students of Professor Gibson, to whom they owed a great deal.

⁂ The Correspondence on the foregoing Papers will be published in the Institution Journal for October, 1939.—SEC. INST. C.E.

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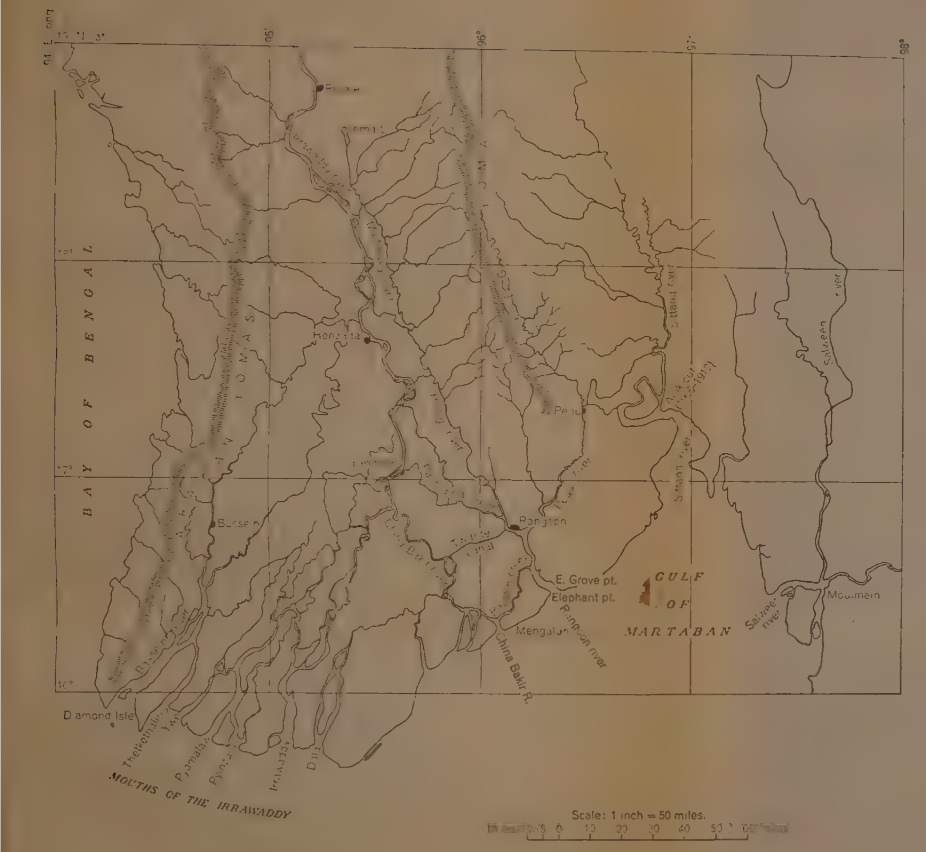
* "The Protection of Dams, Weirs, and Sluices against Scour." Journal Inst. C.E., vol. 10 (1938–39), p. 23. (November 1938.)

† "Construction and Operation of a Tidal Model of the Severn Estuary." H.M. Stationery Office, London, 1933.

‡ "Tidal and River Models." Journal Inst. C.E., vol. 3 (1935–36). (October Supplement, 1936.)

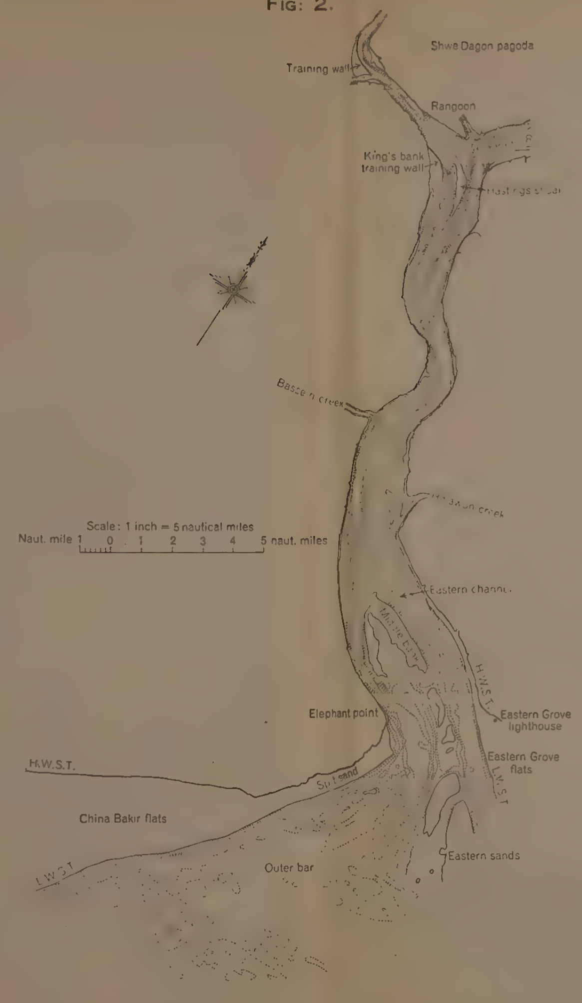
INVESTIGATION OF THE OUTER APPROACH CHANNELS TO THE PORT OF RANGOON BY MEANS OF A TIDAL MODEL.

Fig: 1.



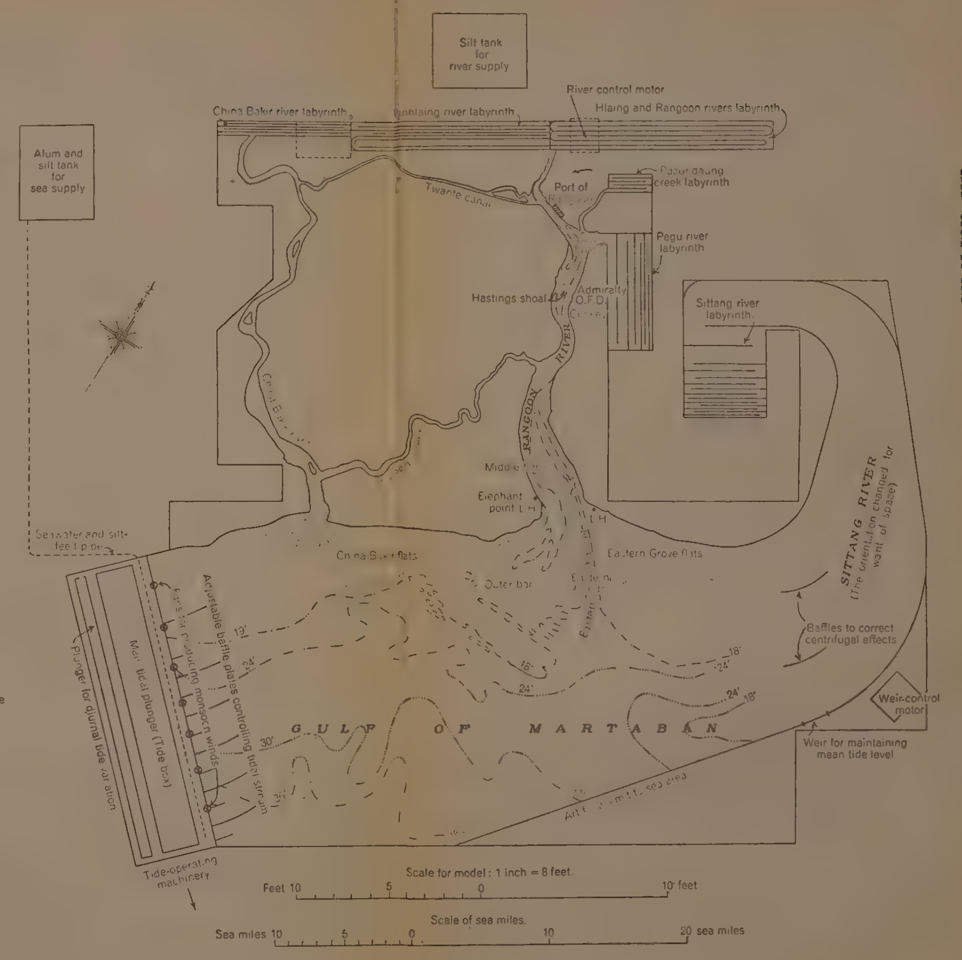
MAP OF IRRAWADDY DELTA SHOWING THE POSITION OF THE RANGOON RIVER.

Fig: 2.



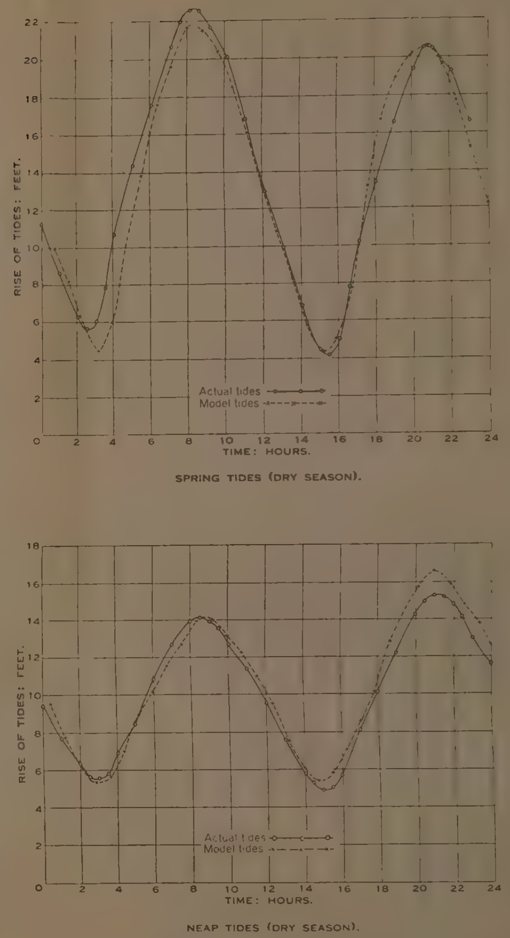
RANGOON AND THE MOUTH OF THE RANGOON RIVER.

Fig: 3.



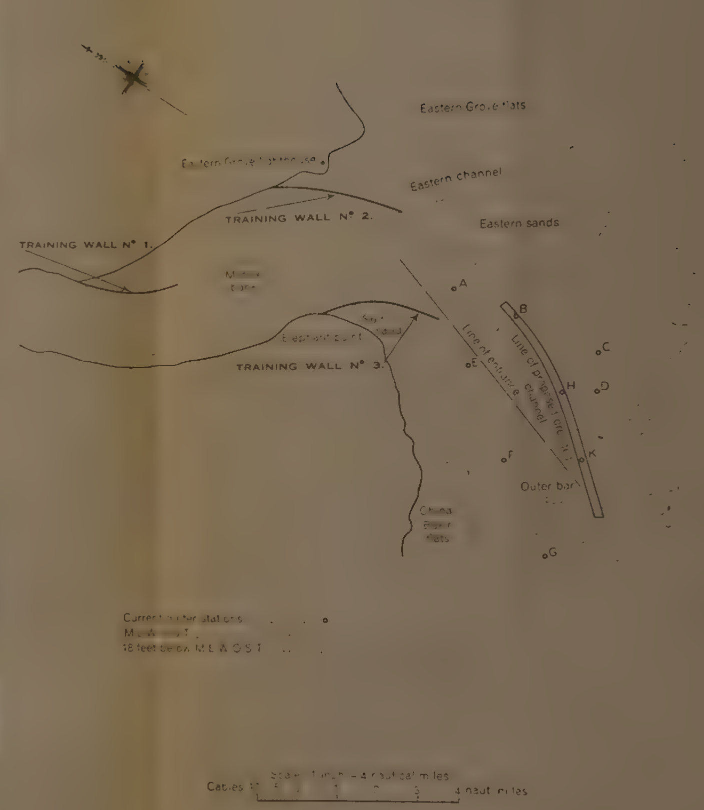
PLAN OF MODEL.

Figs: 8.



COMPARISON OF PRESENT-DAY TIDES AT ELEPHANT POINT, WITH TIDES GENERATED BY MODEL.

Fig. 9.



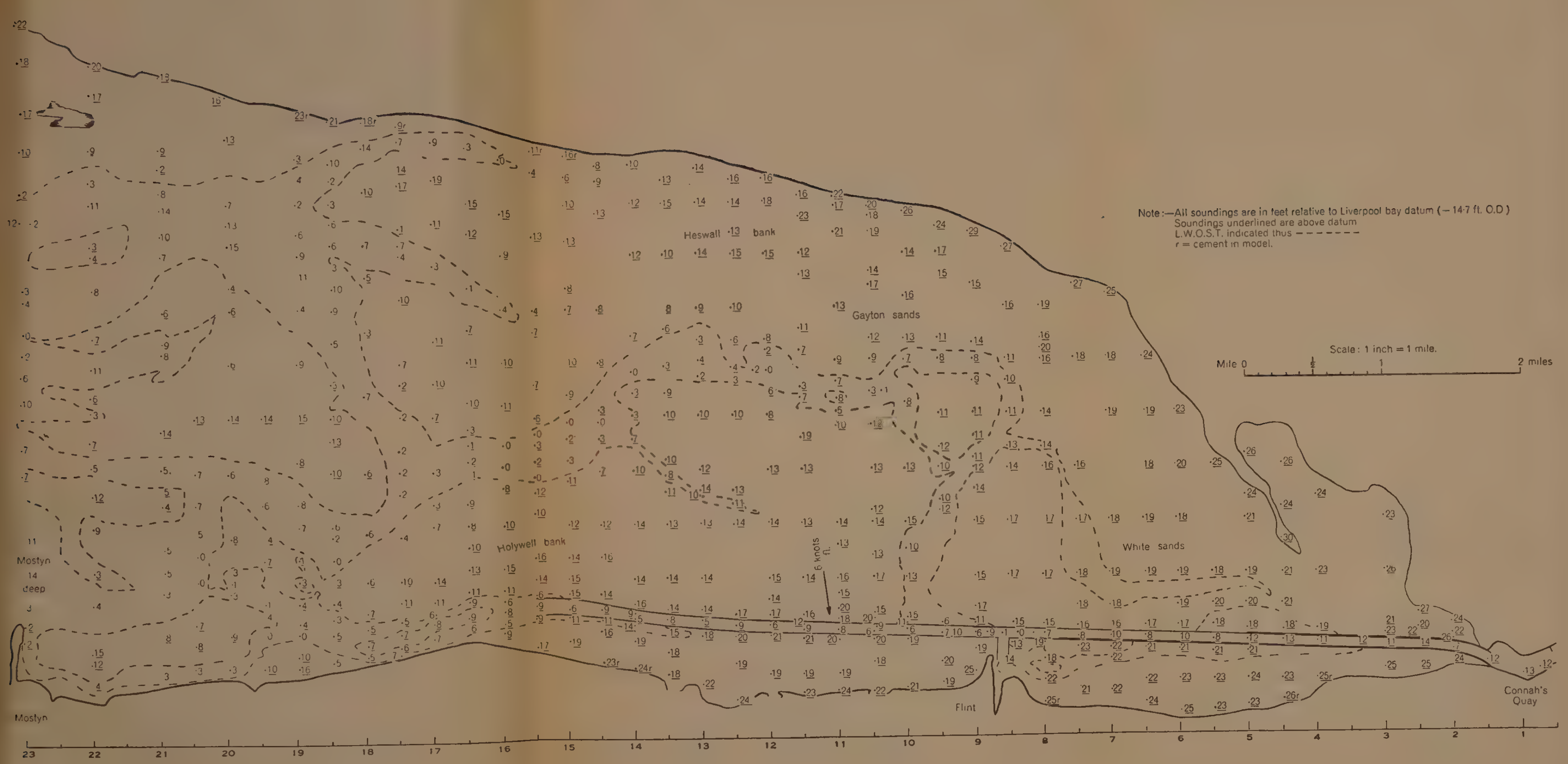
POSITIONS OF TRAINING WALLS TRIED IN MODEL.

OSCAR ELSDEN.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

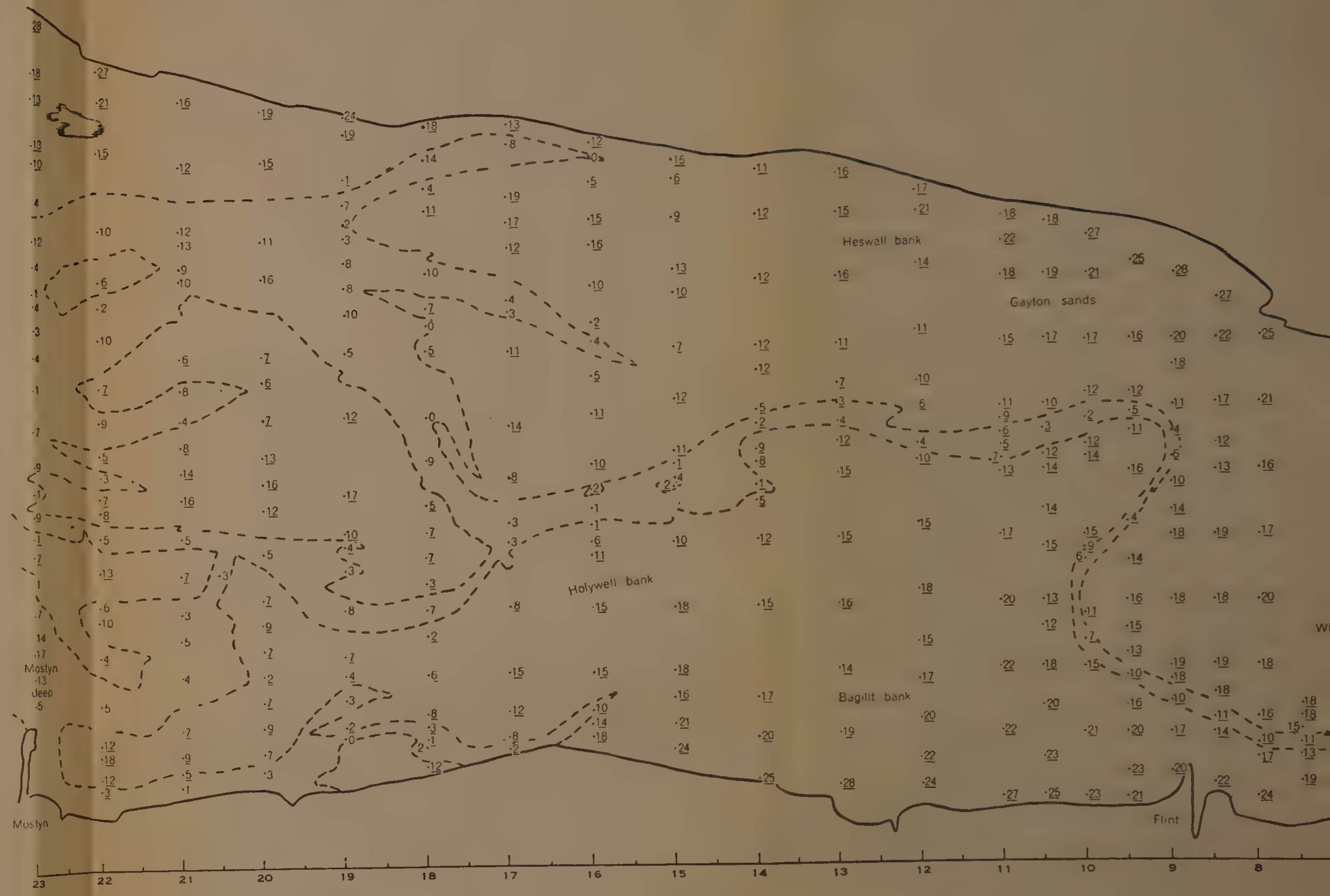
SCHEMES OF IMPROVEMENT FOR THE CHESHIRE DEE: AN INVESTIGATION

Fig. 4.



"BAGILLT WALLS" SCHEME AFTER 9·3 YEARS.

Fig: 5.



INITIAL CONDITION OF RIVER.

William Clowes & Sons, Limited: London.

Note:—All soundings are in feet relative to Liverpool bay datum (– 14·7 ft. O.D.)
Soundings underlined are above datum.
L.W.O.S.T. indicated thus – – – – – –
r = cement in model.

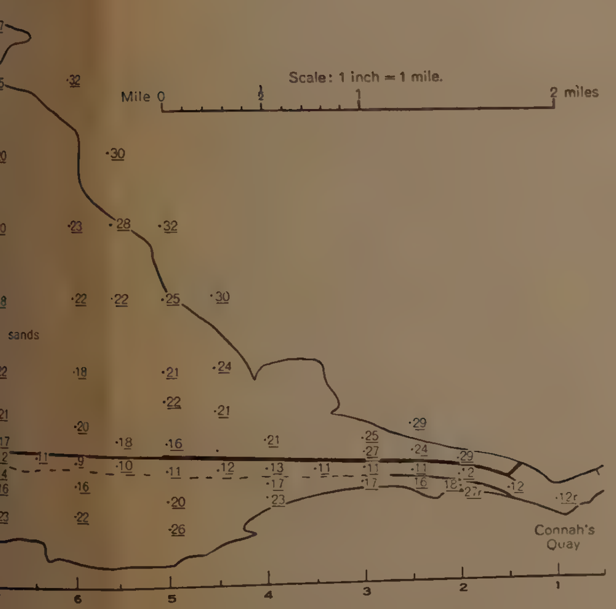
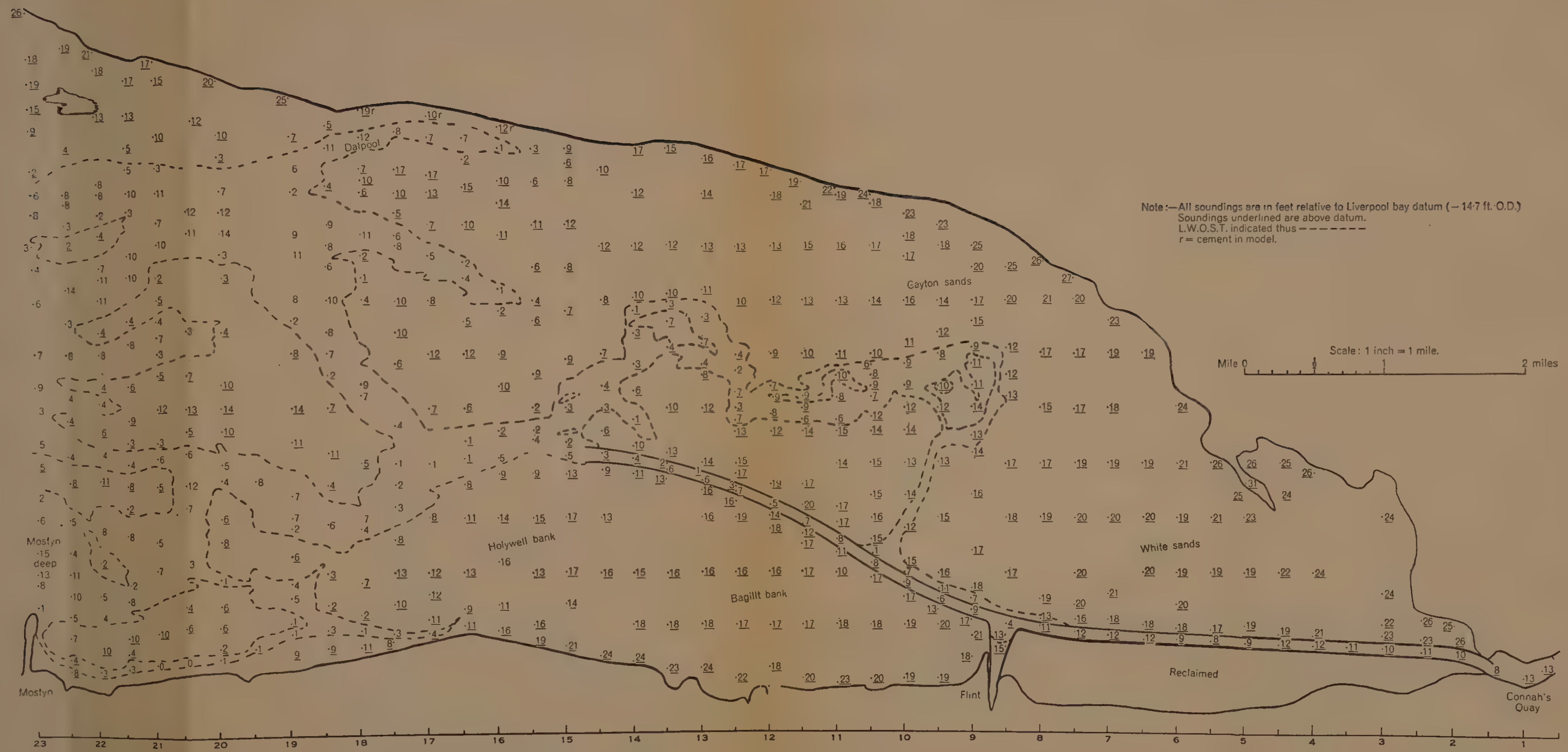


PLATE 2.
SCHEMES OF IMPROVEMENT FOR THE CHESHIRE DEE.

FIG: 8.



SCHEME E AFTER 14·3 YEARS.

J. ALLEN.

ORDINARY MEETING.

25 April, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

It was resolved—That Messrs. F. H. Brunt, S. W. Budd, Robert Chalmers, D. C. Farquharson, A. S. Grunspan, R. W. Mountain, H. C. Ritchie, W. L. Scott, P. J. H. Unna, and J. S. Wilson be appointed to act as Scrutineers, in accordance with the By-laws, of the ballot for the election of the Council for the year 1939–40.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5193.

"The Singapore Airport." †

By Reginald Lewis Nunn, D.S.O., M. Inst. C.E.

TABLE OF CONTENTS.

† Correspondence on this Paper can be accepted until the 15th August, 1939.—Sec. Inst. C.E.

Introduction.

The town of Singapore, with a population of 500,000, stands at the southern point of an island of the same name, in latitude 1° 17′ north, longitude 103° 50′ east, separated from the southern extremity of the Malay peninsula by the narrow straits of Johore. The causeway connecting the island with the mainland has already been described in a Paper * published by The Institution.

Climatic Conditions.

There are no well-marked wet or dry seasons. The average rainfall of 95 inches is spread fairly evenly throughout the year, with some tendency towards a greater precipitation during December than during other months. Normally, rain falls in storms of heavy intensity but brief duration. During the construction of the works described in this Paper, the highest rainfall recorded in a day was 6·2 inches on the 5th January, 1934, and the longest dry period was 22 days from the 22nd January to the 12th February, 1935. Winds are light and variable except between January and March, when monsoon conditions in the China sea cause fresh north-east winds to be prevalent. The strongest gust recorded since the works were begun was 51 miles per hour on the 16th July, 1937; such gusts are usually the forerunners of an intense local rain-storm, accompanied by lightning and thunder of considerable severity. Visibility may be reduced to nil during these storms, but at all other times it is very good, and fog is unknown.

Historical.

Until 1930 there were no air services operating to or within Malaya, and the Royal Singapore Flying Club was the only evidence of civil aviation. This club was inaugurated in 1928 and operated with seaplanes in the harbour owing to the lack of any landing grounds. Towards the end of 1930, the Royal Dutch Air Lines (K.L.M.) instituted a fortnightly air service between Amsterdam and Batavia, calling en route at Singapore. By courtesy of the Royal Air Force these aircraft stopped at the Royal Air Force landing ground at Seletar, which had just been completed. Such an arrangement, however, could only be regarded as temporary, pending the provision by the Colonial Government of a civil airport to serve Singapore. The Royal Air Force landing ground was 40 minutes' drive from the town, and its use by civil aircraft was inconvenient for all.

Selection of Site.

Selection of a site for a civil airport was not an easy matter. Outside

* O. Paterson, "The Johore Causeway." Minutes of Proceedings, Inst. C.E., vol. 220 (1924–25, Part II), p. 250.

the built-up area of the town and suburbs, the island largely comprises rubber-planted hills rising to a maximum height of 581 feet and intersected by swampy valleys and ravines. Towards the centre of the island a fair site was obtainable which could be developed into a landing ground at an estimated cost of $1,800,000 (£210,000 sterling) in a period of from 2 to 3 years. Its distance from both the sea and the town was, however, a material drawback, the Government taking the view that the airport should, if possible, have provision for both land- and seaplanes and be as near the town as possible. While the search for sites was proceeding, Sir Cecil Clementi, G.C.M.G., Governor of the Straits Settlements at that time, suggested that an airport might be provided by reclamation of a tidal swamp within the town-boundaries. The area he had in view was known as the Kallang swamp, covering some 326 acres on the eastern side of the town. This swamp was almost wholly submerged at high tide, whilst at low tide it presented an impassable barrier of soft mud, mangrove, and tortuous channels. Its reclamation would have had to be undertaken sooner or later as a health measure; its development as an airport thus served a dual purpose.

General Considerations.

The Director of Public Works (at that time Mr. George Sturrock, Assoc. M. Inst. C.E.) reported that it was a practicable engineering proposition and estimated the cost of reclamation at approximately $6,200,000 (£725,000 sterling). Instructions to proceed with the scheme were received in May 1931.

Attention was naturally concentrated at first on the extent of the projected reclamation and the means whereby it was to be achieved. Buildings and equipment were to be considered later in the light of anticipated aviation developments.

It was proposed to reclaim the whole of the swampy area on the seaward side of Grove road. On the reclamation a circular landing ground 1,000 yards in diameter would be provided, and other areas would be provisionally earmarked for hangars, etc. A jetty and slipway would be built at a suitable point on the seaward side, with access to the sea by means of a dredged channel. The estuarial discharge of the Geylang river would be canalized past the landing ground. Fig. 1, Plate 1, shows the general locality of the work envisaged.

Surveys of the site indicated that about 8,770,000 cubic yards of filling would be required. These surveys were to a large extent executed from boats at high tide, the mud surface exposed at low tide being too soft and unstable to permit ordinary land-survey methods. In places a pole could be pushed down as much as 40 feet without difficulty.

As a considerable amount of dredging was to be done to form the seaplane channel and anchorage, consideration was given to the question

of reclamation, in part, by hydraulic methods. It was calculated that the dredgings might provide about 30 per cent. of the fill required, but they would be unsuitable in character for incorporation in the landing-ground surface. After reviewing all the circumstances, it was decided to effect the whole reclamation with hard earth "in the dry", the principal arguments in favour of this course being that:

(a) a large organization would be necessary in any case for handling at least 6,000,000 cubic yards of dry filling;

(b) there would be little or no saving in cost, because another coastal reclamation project was in hand which was quite suitable for absorbing these dredgings;

(c) delays of a serious character might occur while waiting for the hydraulic fill to dry out;

and (d) instability might result from the existence of these dredgings as a layer between the old surface and the superimposed dry fill.

Acquisition of Earth Quarry.

Following this decision, steps were taken to acquire suitable hill-land from which the 8,770,000 cubic yards of earth could be obtained. Land already in possession of the Crown, and suitable for this purpose, could unfortunately not be used, because a railway line thence to the airport-site involved a level crossing over a heavily-trafficked street, with no possibility of an over-bridge or a subway. Eventually 150 acres of land were acquired for quarrying purposes at a distance, in a direct line, of about 3 miles from the airport-site. The contours of this land were reasonably good for working as a quarry, the nature of the subsoil was proved by borings, and a rapid survey indicated that enough earth was available above a level which ensured convenient railway gradients.

Physical analyses of the earth gave the following average results:

A. Surface loam with 29 per cent. sand.

B. Subsoil down to about 40 feet: plastic mottled clay with about 24 per cent. sand.

C. Below 40 feet: hard grey clay with 43 per cent. sand.

In a landing-ground surface, which must be capable of withstanding loads of up to 3 tons per square foot in any weather, consolidation and drainage are all-important. A higher percentage of sand than that available would probably have been an advantage for optimum drainage conditions. Nevertheless, exceptionally heavy rains experienced in December 1937, 6 months after the airport was opened, showed that although a certain amount of water lay on the surface owing to the retarding effect of the grass-surface coupled with slight surface irregularities, the weight of heavy air-liners made no appreciable impression and the subsoil drains gave a good discharge. Experience indicates that the class of material was quite suitable.

Quarry Organization.

The quarry-working organization was based on the capacity of the railway conveying the earth to the airport-site. This capacity was estimated at 720 cubic yards per hour, being one train every 10 minutes. To maintain this output from the quarry, a combination of manual and mechanical excavation was adopted. At the outset, five 1-cubic-yard electrically-operated Ruston Bucyrus excavators were purchased at a cost of $110,000 (£12,800). In the later stages, when hand excavation became more arduous and expensive owing to the more compact nature of the earth, an additional four machines of the same type were obtained for $97,000 (£11,300). The labour gangs were mainly employed in surface stripping and slope dressing, and in those areas where ground-contours made mechanical excavation inefficient. The top soil was set aside for subsequent use on the landing-ground surface. At depths below 20 feet the ground was so tightly compacted that it had to be broken up with light charges of explosives. This was particularly the case with class C earth; nevertheless, this was the best quality earth for reclamation. A typical return of the quarry working for the month of March 1936, shows that the output for the month was 180,240 cubic yards (loose in wagon). The cost of working this material was as follows:

Excavation	8·36¢	per cubic yard.
Transport	9·86¢	,, ,,
Reclamation	3·45¢	,, ,,
Staff and general	7·04¢	,, ,,
Total	28·71¢ *	,, ,,

Railway Plant.

Choice of railway plant was influenced by the fact that considerable quantities of suitable plant were just coming on to the local disposals market from the completed extensions of the Singapore waterworks. A quantity of rails and wagons and four steam locomotives were available at once, and it was considered that the 3-foot gauge involved would be suitable for this work. Additional quantities of railway plant were purchased from England and local sources, the full equipment including eight hundred and forty-two 3-cubic-yard "Decauville"-type double-side-tipping wagons, seventeen steam locomotives, and 38,000 linear yards of steel rail. Supplies of rails were supplemented from time to time from the Federated Malay States Railways, which had a surplus stock of 47½-lb. rail.

The ground was practically level between the earth quarry and the airport, but the intervening suburb of Katong caused the railway alignment to follow a tortuous course. The length of double 3-foot-gauge line between the marshalling yards at either end was 3·38 miles. There were thirteen

* The dollar (100 cents) corresponds to 2*s*. 4*d*. in English money.

level crossings, all but one being over roads of sufficient importance to call for colour-light signals in addition to the usual gates. Immediately before reaching the airport-site, the line traversed about 1,000 yards of mangrove swamp on an embankment about 6 feet high. (Reclamation of this swamp has recently been put in hand as a separate project, but utilizing the same organization.) The extensive mud spews which occurred when this bank was being tipped were a warning of what might be expected when making banks on the even softer airport-site. Little trouble was experienced, however, in maintaining the line after it had been established and in use for a month or two. The average time for a train to run between the two marshalling yards was 20 minutes, the schedule being arranged so that, as far as possible, trains crossed each other at the more important level crossings. The $8\frac{3}{4}$ million cubic yards required for the reclamation were conveyed from quarry to airport without difficulty at an average cost of $10\frac{1}{2}$¢ * per cubic yard.

Reclamation Methods.

While plant was being collected and railways were being laid down, a detailed survey of the airport-site was completed. It was apparent that, apart from small areas of comparatively stable ground along the northern fringe, reclamation could not be achieved successfully by merely tipping earth on the saturated and submerged mud. Uncontrollable spews would occur and the earth and mud would become so mixed that the result would be useless for such a purpose as a landing ground. *Fig. 2* gives an impression of the site-conditions during the early stages of the work. It was accordingly determined to bund off areas, and to unwater the interiors so that the mud might dry out and form a crust on which earth could be tipped without causing displacement. The exterior bund had to follow the perimeter of the reclamation ; interior bunds were aligned to suit local factors such as maximum railway curvatures, drainage requirements, ground-contours, etc. Fig. 1, Plate 1, a copy of the contour survey, gives the alignment of the bunds constructed.

Bund construction was a troublesome matter. The trestles supporting the railway continually gave way or turned over and it was impracticable to attempt trestle work extending more than about 150 feet at one time. In addition, the earth tended to break up and mix with the mud, which spewed and moved in such a way as to suggest that in some places stability would be unattainable. These tendencies were, however, successfully overcome by the expedient of dredging a channel on the line of the bund immediately in front of the trestle work. Trestling was thus rendered more difficult, but the earth kept together more readily, sinking as one solid mass into the mud. Subsequent borings showed that penetration of the earth reached a depth of more than 40 feet in places. The

* Footnote (*), p. 73.

Fig. 2.

SITE-CONDITIONS DURING EARLY STAGES OF WORK.

Fig. 3.

BUND CONSTRUCTION AND DRYING-OUT PROCESS.

Fig. 4.

FILLING IN PROGRESS IN THE PULAU GEYLANG AREA.

Fig. 6.

LOADED LORRY TESTING CONSOLIDATION OF LANDING GROUND.

average quantity of earth required for 1 linear yard of bund was 168 cubic yards. In some sections a bund developed a tendency to turn or creep sideways. When this occurred work was suspended for a few months and the side pressure relieved by dredging. As soon as an area was encircled by a bund, a pump was installed at a convenient point and the area unwatered as shown in *Fig. 3* (facing p. 74). Herring-bone drainage channels dug in the mud surface and leading to the pump sump soon resulted in drying out and crusting of the mud. A minor complication arose here from the creation of admirable breeding places for mosquitoes in the countless waterfilled cracks. Oiling and liming kept this in check.

The drying-out period ranged from a few weeks to several months, according to weather and other circumstances. The filling-in of an area then presented no difficulty; earth was placed in 2-foot layers and consolidation was effected by caterpillar tractors. It was ascertained by borings that a depth of 12 feet of filling depressed the mud surface approximately 2 feet.

A governing factor in the rate of progress was the bunding and drying-out process. With the object of maintaining full train-schedules, reserve tips were kept where any accumulation of trains could be shunted and unloaded. To facilitate train handling on the reclamation, visual signalling was employed between the various tip-banks and an observation tower adjacent to the marshalling yard.

The disappointment caused by Pulau Geylang may be mentioned as indicative of some of the unusual features of this work. This island lay within the perimeter of the landing ground and it was planned to incorporate it in the bunding system. It appeared, moreover, at first sight that here was a valuable piece of reclamation already made, as an active sawmill had occupied the island before it was acquired by the Government. When, however, the buildings and machinery had been removed, it was found that the whole island was composed of sawdust and timber in various stages of decay. The whole of this unstable material had to be excavated to a depth of 14 feet and taken away by train to a dump clear of the landing ground. Filling in progress in this area is shown in *Fig. 4*.

Landing-Ground Drainage.

In order to throw off surface-water and to provide a suitable gradient for the subsoil-drainage system, the landing ground was finished in the form of a dome. The reduced level at the centre is 115·31, whilst the perimeter level is 107·50. Tide records prior to the airport construction showed a highest tide-level of 104·50. A margin of 3 feet was considered an adequate freeboard and sufficient to secure discharge from the drainage system. An additional foot in the height of the reclamation would have called for about 500,000 cubic yards more filling, and on the score of cost it was therefore essential to keep the finished level as low as was consistent with safety.

Since the work was completed, an exceptionally high tide at level 106·23 was recorded on the 20th December, 1937, but no damage was caused thereby. With a circular landing ground the most practical way of meeting drainage requirements is by doming. From a pure flying point of view a flat surface is the best, and it was judged that a gradient of 1 in 190 was the flattest which would satisfy minimum drainage requirements. A steepening of the gradient, whilst improving drainage, would have three disadvantages:

(*a*) an aeroplane on one side of the ground might not be observed from another on the opposite side,

(*b*) take-off and landing conditions would not be so good, and

(*c*) costs would be higher on account of the extra filling required.

The doming adopted in this case called for about 1¼ million cubic yards of earth fill above perimeter-level.

Fig. 5, Plate 1, illustrates the subsoil-drainage system. The 4-inch tile pipes are laid from 3 feet to 4 feet deep on lines 50 feet apart. They are bedded on 3-inch broken stone, the top half of the joint being overlaid with a strip of nipah leaf and clay. The discharge of these drains was delivered by intercepting lines of glazed stoneware pipes into the perimeter drain or direct into the sea.

Landing-Ground Surface.

While the reclamation was in progress experimental grass-nurseries were cultivated. Top soil had been set aside at the quarry for use on the landing ground, but it was of poor quality and manuring would have been required. The grass eventually selected was *Axonopus compressus* and 19½ acres of this had been cultivated by the time the landing ground was ready for turfing. The procedure then adopted was to cut 12-inch square sods from this nursery for conveyance to the grassing gangs. Shallow holes were made by light crowbars at 8-inch centres, and into each hole was put a small tuft broken off from a sod together with a mixture comprising 1 part decomposed vegetable matter and 2 parts cattle manure. One 12-inch square sod would thus be made to cover about 1¼ square yards. Within a few months the whole surface was well covered with a good carpet of grass, after which final rolling was carried out. Premature rolling would have compacted the soil too tightly for the roots to penetrate. After heavy rains it is found that large air-liners are liable to tear the grass by sharp application of brakes during a landing run. Tail-skids also cause damage, but, fortunately for aerodrome proprietors, the tail-wheel is replacing the tail-skid in all large modern aircraft and in many light aircraft as well. It has not been found necessary in Singapore to adopt the practice, in force at some aerodromes in other countries, of charging a higher landing fee for tail-skid aircraft. *Fig. 6* (facing p. 75) shows a loaded lorry testing the consolidation of the landing ground prior to grassing operations, whilst *Fig. 7* is a photograph taken from the air showing the reclamation com-

Fig. 7.

R.A.F. Photograph—Crown Copyright reserved.

RECLAMATION OF LANDING GROUND COMPLETED.

Fig. 11.

ENTRANCE TO AIRPORT FROM GEYLANG ROAD.

Fig. 15.

ADMINISTRATION BUILDING.

Fig. 18.

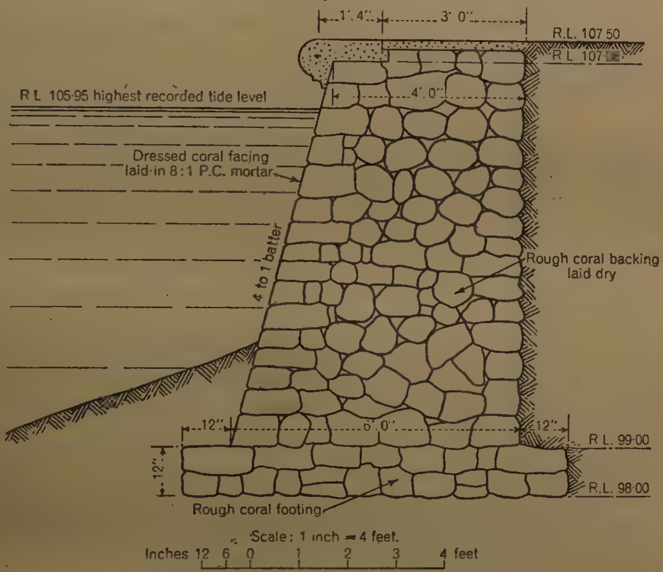
GENERAL VIEW OF ADMINISTRATION BUILDINGS, HANGARS, SLIPWAY AND JETTY.

pleted, the dark areas in the landing ground itself having already been grassed over. This photograph also shows the relative positions of the airport and the town of Singapore.

Retaining Wall.

The design of the retaining wall along the waterside limits of the reclamation required careful consideration. Fortunately, supplies of coral were readily available in adjacent islands, and could be brought by *tongkang* to the site at an average cost of \$1·15 (about 2*s*. 8*d*.) per cubic yard. This material was particularly suitable because of its extreme lightness, a

Fig. 8.



Section through Coral Retaining Wall.

dry cubic foot weighing only 73 lb. A wall built in coral blocks was therefore unlikely to induce sufficient loading to disturb the stability of the earth bund on which it was built. Minor movement or displacement can be met by adjusting the alignment of the pre-cast concrete coping blocks with which the wall is capped. *Fig. 8* shows a section through the wall, which cost \$7·00 (about 16*s*. 4*d*.) per linear foot.

Adjacent to the slipway and jetty the toe of the wall was protected by concrete sheet-piling.

Seaplane Channel and Anchorage.

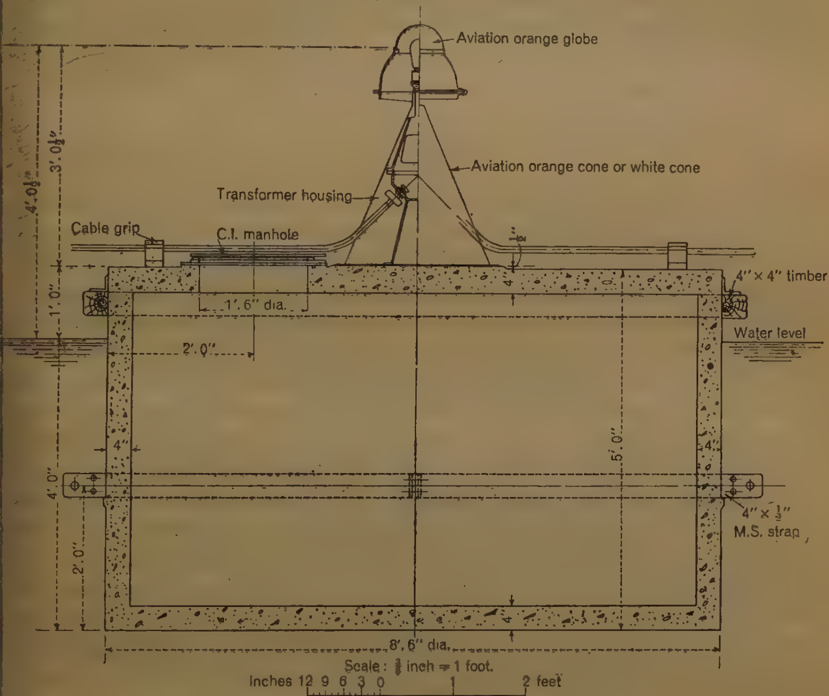
The dimensions and alignment of the seaplane channel and anchorage were dictated by physical surroundings. The modern flying-boat demands

a clear run for take-off of about 1½ mile with free approaches. Such a distance is available for any wind-direction in the open water outside Singapore harbour, and the channel was therefore primarily devised to give access between that open water and the slipway and moorings. In practice an " Empire " type flying-boat has on a few occasions landed and taken off within the seaplane channel itself, but the masts of *tongkangs* and other craft in the neighbourhood constitute too dangerous an obstruction for this practice to be regularized. The light seaplanes of the Royal Singapore Flying Club can normally operate within the channel without difficulty. The channel width of 200 yards was the maximum obtainable without obstructing marine craft plying up the Kallang river on the one hand and to the wharves along Tanjong Rhu on the other. Experience has shown that this is a comfortable width for the purpose, and no difficulty has been experienced in manœuvring any seaplane or flying-boat in the channel. A minimum depth of 6 feet at extreme low tide was considered necessary, and to allow a margin the channel was dredged actually to 7 feet 6 inches. Out of approximately 2,150,000 cubic yards dredged from the sea-bed for this channel, about one-third was used in the coastal reclamation referred to earlier in this Paper, the remainder being dumped at sea. The grab-dredgers were employed mainly in pulling up old piled foundations of Malay villages which straggled across the line of the channel. Part of the area had been used for many years as a dumping ground for timber logs brought in by *tongkangs* from other seaports prior to being rafted up to the river Kallang and other channels leading to sawmills. A number of these logs had been abandoned by the owners after they had become submerged in the mud, and their removal constituted a further considerable task for the grab-dredgers. It was, moreover, vitally important to leave no such snags behind as a partially freed log in the seaplane channel might bring disaster to a flying-boat. The material to be dredged was mainly soft mud, but one patch of shale was encountered and there was also a sand spit to cut through at the west end of Tanjong Rhu promontory.

It has not yet been possible to assess the amount of recurrent dredging that will be necessary to maintain this channel but, after 9 months of operation, soundings reveal only a small quantity of silting. To protect aircraft from the menace of floating timber and other debris and to facilitate the control of small craft, the sides of the channel have been demarcated by two floating timber booms. These are kept in place by concrete piles at 85-foot intervals and are held together by steel-wire rope and chains. The logs used for the purpose are known as *Geronggang*, the average dimensions being 24 inches in diameter and 16 feet long. It is too soon yet to say what life these logs will have. Chains were not available when the booms were first constructed, and 3-inch-circumference continuous steel-wire rope was used to hold the logs together. It was not long before the seaward ends of the booms were damaged. A succession of heavy seas produced a rolling

and sawing motion which frayed and twisted the wire rope, and then a number of *tongkangs* crashed through and completed the destruction. Within the sheltered water behind Tanjong Rhu, the booms have not given much trouble, but it is clear that wire rope will not be satisfactory in the seaward sections liable to considerable wave motion. Experiments are proceeding with various types of chain fastenings. Reinforced-concrete

Fig. 9.



Reinforced-Concrete Buoy.

buoys 100 yards apart and surmounted with orange lights serve to indicate by night the alignment of the timber booms. *Fig. 9* illustrates one of these buoys. The entrance to the channel from the sea is marked by two concrete piled beacons carrying " Aga " lights.

Layout and Equipment.

In 1934, when the reclamation work was well in hand, the Author was deputed by the Straits Settlements Government to examine airport layouts and aviation developments in other countries with a view to determining the most suitable layout for Singapore. In a period of 4 months a rapid tour was made through the United States of America, European

countries, Africa, and India. Airport layouts and equipment, wireless services, airline operations, legislation, and administration were studied[1]. The building layout subsequently adopted for Singapore attempted to incorporate the advantages and avoid the drawbacks observed as features of other airports. Of first importance was the grouping of buildings in such a manner as to minimize them as aerial obstructions without detracting from their effective utility. Fig. 10, Plate 1, illustrates the layout adopted. The road-entrance to the airport is in the direction of the heart of the town and the main buildings lie on an axis of infrequent wind-direction. In effect, aircraft can take-off or land into wind on any bearing without having to pass over any of these buildings.

Hangars.

The dimensions of the two hangars were determined as a result of a survey of the probable types and numbers of aircraft likely to use this airport; they have a floor area of 300 feet by 150 feet and a clear height of 35 feet. To facilitate handling of aircraft into and within the hangars, each is provided with one end and one side door 150 feet in width. The detailed design of the hangars was prepared in the Chief Engineer's Office of the Crown Agents for the Colonies. Allowance was made for a wind pressure of 15 lb. per square foot, and also for suspension of weights of up to 1 ton from forty-two points in the roof-trusses. Side sheeting is in No. 20 B.W.G. corrugated iron above a 6-feet brickwork plinth, and roof-sheeting is in ⅜-inch corrugated asbestos. The foundations comprise 16-inch by 16-inch pre-cast reinforced-concrete piles ranging in length from 55 feet to 90 feet. They were driven to a set of ⅜ inch for the last ten blows of a 3-ton hammer falling 3 feet. There are one hundred and fifty-four piles per hangar and the maximum load on a single pile is 40 tons. Floors are paved with pre-cast concrete slabs laid to a fall of 6 inches from the centre to the sides. Hangar annexes 20 feet wide with ceiling heights of 19 feet 6 inches on the ground floor and 13 feet 4½ inches on the first floor, provide workshop and office accommodation respectively for aircraft owners and operators. Ten fireproof doors 6 feet wide give access between hangar and workshop, and there are interior windows upstairs to afford observation into the hangar from the offices. The orientation of the hangars so that the annexes faced the entrance road permitted a certain amount of architectural treatment, as shown in *Fig. 11* (facing p. 76).

Administration Building.

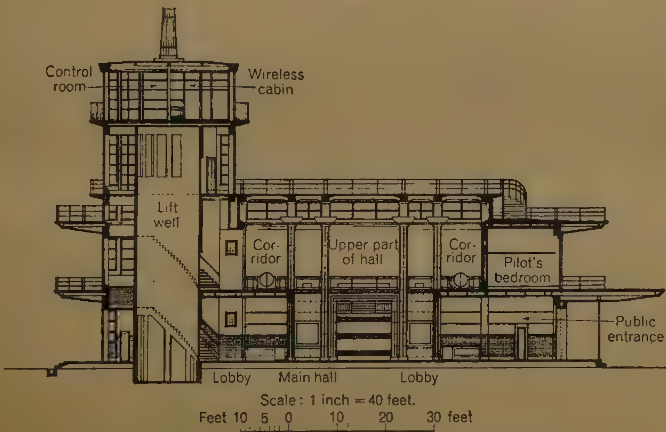
This building (*Figs. 12*, *13*, and *14*, pp. 81 *et seq.*) provides accommodation on the ground floor for operating companies' offices, post office,

[1] "A Report on Civil Aviation", recording the Author's impressions and experiences, may be seen in the Institution Library.—Sec. Inst. C.E.

medical and customs examination, restaurant, telephone booths, money changer's booth, lavatories, and a communication office at which departing and arriving pilots report. On the first floor are offices for the airport manager, aircraft inspector, and meteorological department, also bedrooms and a sitting-room for pilots, and a dining-room with roof verandah. An electric lift gives access to the first floor and to the flat roof which is useful and popular as an observation terrace for the public. From this level the control room at the top of the tower is reached by an internal spiral stairway. The control room with its encircling balcony has an unrestricted view over the whole airport.

Its equipment comprises a switchboard for all the airport lighting, loudspeaker, and other signalling facilities (Verey pistols, Aldis lights, and flags), chart tables, etc., for the use of the control officer. These are

Fig. 12.

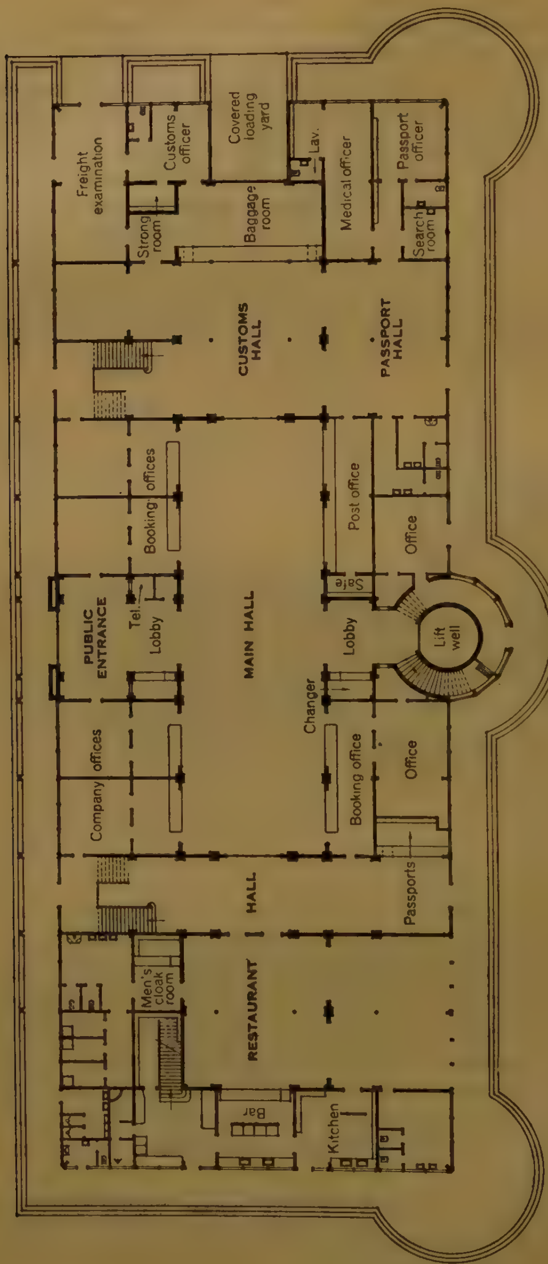


ADMINISTRATION BUILDING: CROSS SECTION.

suitably arranged in positions overlooking the landing ground, whilst the remaining floor area is occupied by the wireless operator and his equipment. The loudspeaker installation enables the control officer to issue directions to persons on the pavement immediately in front of the building in sufficient volume to be heard above the noise of aircraft engines. The control room is enclosed in $\frac{3}{16}$-inch non-actinic glass, with plain glass panels in selected sectors. Glass-sheeted control towers elsewhere having been noted as very hot, the roof of this control room was covered with 2-inch thick cork slabs laid in asphalt. In practice this control room has proved agreeably cool at all times.

Wireless reception is effected by local aerial, whilst transmission is by remote control to a station at Paya Lebar, about $2\frac{1}{2}$ miles to the north-east. The wave-lengths in use are 900 and 45·5 metres. A Marconi-Adcock direction-finding station was built about $1\frac{1}{2}$ mile north

Fig. 13.



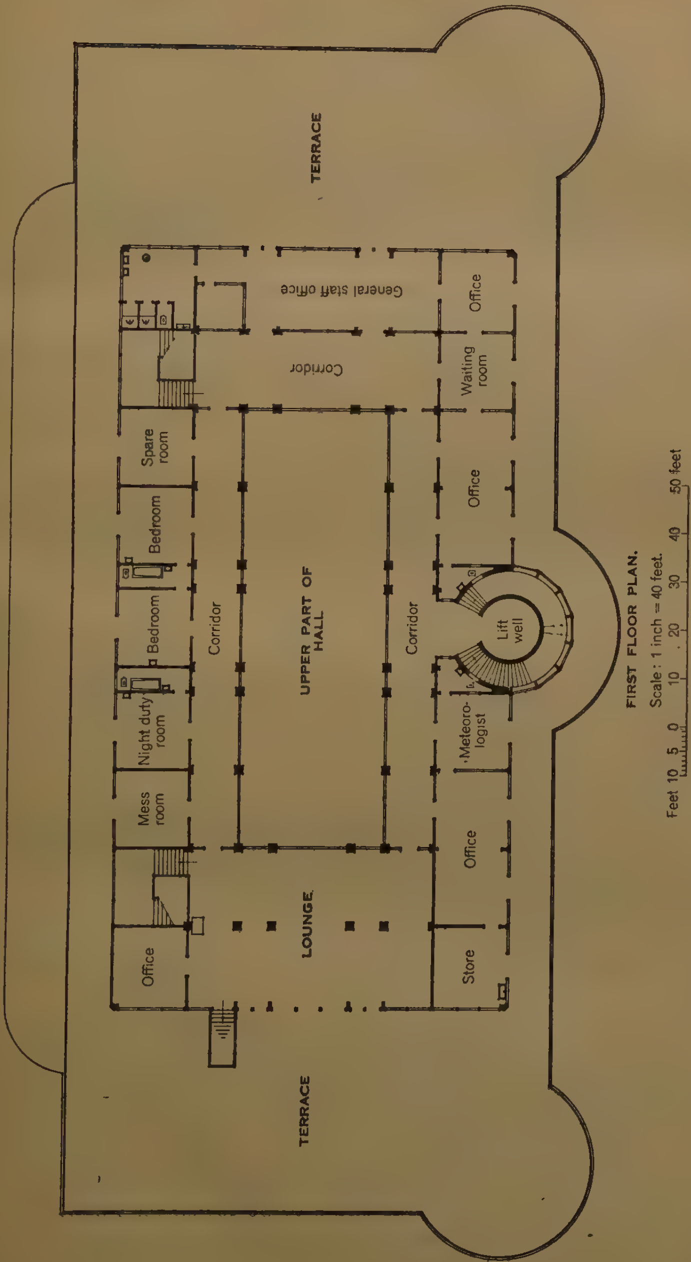
GROUND FLOOR PLAN.

Scale: 1 inch = 40 feet.

Feet 10 5 0 10 20 30 40 50 feet

ADMINISTRATION BUILDINGS.

Fig. 14.



FIRST FLOOR PLAN.

Scale: 1 inch = 40 feet.

ADMINISTRATION BUILDING.

of the airport—the nearest site which met requirements affecting correct reception. This station works on medium wave, and the bearings are sent by direct telephone-line to the control room. Calibration within 1 degree has been achieved. It had been feared that automobile ignition-systems might interfere with reception, but fortunately this has not been the case.

The general appearance of the administration building is indicated in *Fig. 15* (facing p. 77). It is carried, like the hangars, on pre-cast reinforced-concrete piles. A total length of 14,332 feet was used in two hundred and fourteen piles, the majority of which were of 16-inch by 16-inch section. The superstructure comprises a steel framework with reinforced-concrete slabbing and brick panelling. Most of the exterior and interior finishes are in "Lap" slabs, with stainless steel and aluminium strips. Floors are finished in rubber tiles and staircases in cork tiles, with English glazed-tile dados. The wearing surface of the roof and terraces is buff "Colorcrete" mortar superimposed on three layers of "Rok" and asphalt. In the restaurant the floor is finished in "Spuncrete" polished tiles, and the walls in "Vitrolite" glass.

Two telescopic canopies, one each for arrivals and departures, give protection from weather between the building and the aircraft. These have proved very popular adjuncts.

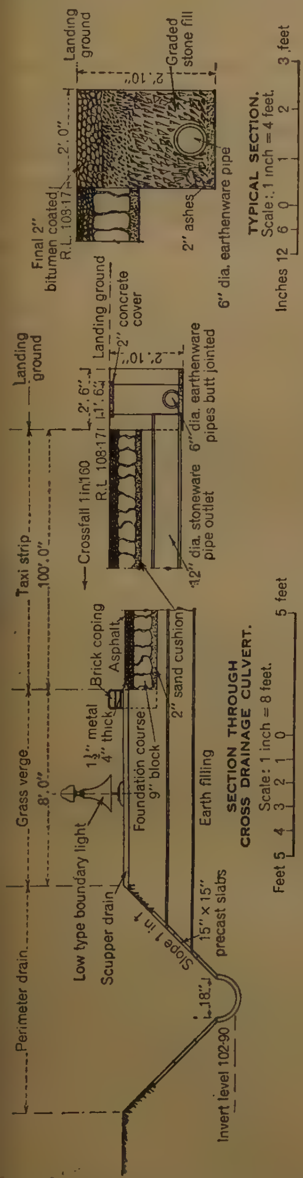
Paved Areas.

The paved area, or "tarmac" as it is termed, is constructed with pre-cast concrete slabs of the same pattern as those laid in the hangars. A percentage of slabs is provided with "keyholes" to facilitate lifting and adjustment if necessary. The surface is graded in slight falls to underdrains which are spanned by perforated slabs. After allowing a period for settlement, the joints between slabs were filled with a mixture of "Mexphalte" and sand. The cost of this paving was \$1·97 (about 4*s*. 8*d*.) per square yard. Immediately in front of the administration building special slabs were used incorporating ½ inch of green "Colorcrete" with a view to mitigating glare, but the experiment has not been an unqualified success as the "Colorcrete" portion tends to flake away.

In order to reduce wear on the grass surface immediately in front of the "tarmac", two taxiways in asphalt-macadam, 100 feet wide, extend along the perimeter of the landing ground for a distance of 1,100 feet in either direction. *Figs. 16* is a typical cross-sectional sketch of these taxiways. One of these taxiways is connected by a bridge over the perimeter drain to the concrete-slabbed area at the head of the seaplane slipway. This enables seaplanes and flying-boats to be towed on trolleys from the slipway to one of the main hangars. It also allows an amphibian aircraft to transfer from land to water and vice versa. Ultimately, if there is sufficient demand, a separate hangar for seaplanes may be constructed

near the head of the slipway. The cost of the asphalt-macadam taxiways was \$2·10 (about 4*s.* 11*d.*) per square yard.

Figs. 16.



Section Across Taxiways, showing Outlet to French Drain.

Slipway and Jetty.

The slipway is 100 feet wide, and is sloped at 1 in 15. Its length of 263 feet provides for a minimum depth of water of 8 feet 6 inches at the foot of the slipway. Figs. 17, Plate 1, illustrates this work. When the design was prepared, no information was available regarding future weights and dimensions of flying-boats or as to the methods to be used by their operators for beaching. Test-piles showed the ground here to be even worse than under the hangars, and it would have been a most expensive proposition to build a slipway which would carry a heavy concentrated load at any point on its surface. It was accordingly decided to allow for loads of up to 60 tons to be carried along defined lines only (with rails imbedded in the surface) and to allow for a distributed load of 200 lb. per square foot elsewhere. It has subsequently transpired that the new "Empire" type flying-boat uses a beaching chassis of twin-tired wheels with a wheel-gauge which matches the rail-gauge closely enough for practical purposes. The piled foundation of this slipway comprises one hundred and thirty 16-inch octagonal reinforced-concrete piles ranging between 67 feet and 129 feet in length. (The aggregate pile-length was 12,460 feet.) With a view to weight-saving, the underside of the slipway-slab was cast in cellular form; below L.W.O.S.T., however, a plain slab was used.

The jetty adjacent to the slipway measures 300 feet by 35 feet in plan and is designed to provide access for passengers, mails, and freight, and also a platform for a travelling crane. As will be seen from Figs. 17, Plate 1,

the pile foundations are protected by concrete cylinders. One of the two sets of landing steps has been provided with a canopy to protect passengers from the weather between launch and motor car. Whether travelling by aeroplane or flying-boat, therefore, the passenger is at all times protected from the weather—a feature noticeably absent at many airports and at most shipping wharves. Consideration is being given to the provision of a floating landing stage connected by covered gangway to the jetty. More experience is, however, first required of the behaviour of flying-boats in a strong cross wind. The crane on the jetty is electrically operated and has a capacity of 2 tons at 60-foot radius. Both the slipway and jetty were constructed "in the dry" within an earth bund, which was afterwards removed by dredgers. *Fig. 18* (facing p. 77) shows the relative positions of the slipway and jetty, with the administration building and hangars in the background.

Lighting Equipment.

Energy is obtained from municipal mains by underground cable to a switch and meter room near Grove road. Duplicate 0·3-square-inch four-core cables are each capable of carrying approximately double the initial load at the airport. As the Municipality charges different rates for lighting and power, two sets of mains beyond the meter room were necessary.

Two underground ring-mains, each 0·1-square-inch four-core cables, were laid past the administration building, returning behind the hangars to the meter room. From these ring-mains lighting and power services were laid to all buildings, road-lights, etc. An additional cable of similar section was provided as a standby against faults occurring under paved areas.

The services provided include :—

(*a*) Lighting of landing ground and seaplane channel.
(*b*) Lighting of airway obstructions.
(*c*) Lighting, ventilation, lifts, etc., in the administration building.
(*d*) Lighting, ventilation, etc., in hangars.
(*e*) Lighting, ventilation, etc., in subsidiary buildings.
(*f*) Lighting of approach roads.
(*g*) Lighting and power at the jetty.
(*h*) Loudspeaker equipment.
(*i*) Lighting, ventilation, etc., at the Royal Singapore Flying Club.
(*j*) Lighting, ventilation, etc., for offices, workshops, etc.

Landing-Ground Lighting.

This follows the general principles enunciated in British Standard Specification No. 563/1934.

On the roof of the control tower there is a red neon hairpin-tube beacon, which can be operated either as a steady light or as a flashing beacon denoting the letter "S."

The perimeter of the landing ground is indicated by twenty-nine boundary-lights spaced at 100-yard intervals. These are of the cone type, so that in addition to the aviation orange emission of light from 5 degrees below horizontal to the zenith and through 360 degrees in azimuth, a white lighted cone shows round the stem of the fitting and gives a sense of perspective to the pilot. The lamps used take 6·6 amperes at 6·6 volts, and they are supplied through a transformer in the ground under each light. The stems of the fittings are provided with weak nipples so that they easily break off in the event of being hit by an aircraft. Special measures were necessary to guard against condensation in the transformer-boxes. Energy is supplied in series through a single-core 0·01-square-inch underground cable ringed round the ground and terminating in the switch-room in the control tower. A transformer is interpolated here to step up the pressure to a suitable figure. The height of the boundary lights is 3 feet 6 inches, except for those along the taxiways, which were kept down to 2 feet 6 inches so as to be below the wings of aircraft.

Six floodlights are situated at approximately equal intervals round the landing ground. Two of these are 30 yards back from the perimeter, whilst the remainder had to be on the perimeter itself. Each has a beam candle-power of approximately 1,000,000. In practice, a pair is normally used and the illumination thus provided is of the order of 0·15 foot-candle. Operation is by switch in the control tower. Lamp-bulbs are of special tubular pattern with adjustable focussing mirrors and are arranged in three banks of two each, giving an aggregate of 6 kilowatts. Each floodlight is housed in a special hut with walls and roof of "Uralite" and a 180-degree glass window. Fresh air is introduced into the hut by a fan which operates automatically when the light is turned on; a flow of fresh air is necessary to guard against excessive condensation, due to the considerable heat generated by these lamps. Five of these floodlights are fixed in position facing the centre of the ground, whilst the sixth is fitted on a turntable so that it can be turned to illuminate the seaplane anchorage when necessary. The best height above ground-level for these floodlights was determined as a result of night-flights carried out by the Author with several lights experimentally set at different levels. With a height of 14 feet for the central focal plane it is found that sufficient light escapes past the centre of the domed landing ground to illuminate an adequate area, whilst ensuring that the highest intensity is at that point where a normally piloted aircraft would touch down. The overall height of the huts is 18 feet.

Seaplane-Channel Lighting.

Thirty-one concrete buoys each carry a light of landing-ground boundary type. There are two separate circuits on each side of the channel and all lights are controlled from the switchboard in the control tower. The submarine cable comprises one conductor of fifty-one strands of tinned

copper wires each 0·08 inch in diameter, insulated with "Maconite" rubber compound of 0·1 inch radial thickness, armoured with a layer of 7/0·029-inch galvanized-steel strands, and finally sheathed with resilient tough "Maconite" compound of 0·15 inch radial thickness. Unfortunately, the local *tongkang* owners appear to think the cable has been provided as a better holding place for their anchors than the channel bottom, and breakages have been rather frequent. Discouraging measures are being taken.

Obstruction Lighting.

These are fitted in pairs on all main buildings, wireless masts, flood-light housings, etc., also on certain high chimneys and the flagstaff of the Swimming Club at Tanjong Rhu. Control is mainly by time-operated switches.

Wind-Tee Lighting.

A standard-pattern 20-foot by 20-foot wind-tee is illuminated by forty 15-watt white lamps, with a switch in the control tower. To indicate the position of the tee to observers at or near ground-level, green, red and white lights arranged in navigation manner have been suspended under the tee.

Administration-Building Lighting.

To conform with architectural features the internal lighting of the main hall and public rooms is effected by white opal and orange tubular-strip lamps, supplemented by lights concealed in large bowls near the top of the main pillars. Wiring was concealed, and in order to facilitate repair it was carried in a hard rubber conduit made locally. Rubber was chosen in place of the more usual steel, because of condensation troubles. Careful attention was given to the siting of fans in those areas likely to be crowded at times. In the lift well, all wiring is provided with metallic covering and contacts are fitted with chokes and condensers to obviate wireless interference. A greater degree of silence is secured by the elimination of the usual mechanical contactors for floor switch-control, and the operation of this through a magneto system and induced current and telephone-type relays. The "tarmac" immediately in front of the building is illuminated by small floodlights fixed on the balcony of the control tower.

Hangar Lighting.

Interiors of hangars are lighted by means of mercury-vapour gaseous discharge lamps, the energy for these being transformed down to 110 volts. Socket outlets for power and light are suspended from the roof-girders at intervals and can be raised or lowered by winches on the walls. Three special projector-lanterns are provided on telescopic portable standards

by means of which "spot" illumination for aeroplane repairs can be provided at any angle and height up to 20 feet. Areas outside the hangars are lit by small floodlights situated above the doors.

Approach-Road Lighting.

Mercury-vapour discharge lamps are used along the approach road, and there is a red neon sign at the entrance gates.

Smoke Pot.

In addition to the wind-tee and the usual wind-socks, an indication of wind direction and strength is given by a smoke pot located in a manhole in the centre of the landing ground. This smoke pot is electrically operated and is controlled by a switch in the control tower. The principal elements in the apparatus comprise an electrically-driven air-compressor, an air receiver, an oil container and an atomizing jet, and an electrically-heated hot plate. When the control switch is closed, current is turned on to the hot plate and the air-compressor motor. Upon the air attaining a predetermined pressure, a release valve is opened and allows oil to flow to the jet, where it becomes atomized and thrown on to the hot plate. The resulting dense white vapour escapes through a hole in the manhole cover into free air. The time taken to produce the smoke after closing the switch is about 5 minutes; no attention is required other than a daily visit to replenish oil, etc., and the apparatus has been completely successful.

Ancillary Buildings and Equipment.

The premises for the Royal Singapore Flying Club comprise offices and changing rooms on the ground floor, a lounge with verandah and bar, a committee room over the porch on the first floor, and a roof balcony above. It was necessary to carry this building on concrete piles about 60 feet in length.

A fire station and storeroom is located on the side of the taxiway leading to the slipway. This building, the flying club, and a floodlight are grouped together so as jointly to form only one obstruction on a line of approach.

The fire engine is of the latest type, providing three methods of fire fighting. There are four 60-lb. carbon-dioxide cylinders interconnected to a rubber hose with a long branch pipe and distributor. The carbon dioxide is for immediate use to facilitate extricating personnel. The fire is then attacked by foam produced in Merryweather's patent "Xaust suds" apparatus capable of supplying 1,000 gallons of foam. In addition, the same pump driven from the chassis engine can be used for delivering an ordinary water jet at a pressure of 150 lb. per square inch.

Two blocks of garages on rafted foundations have been provided in suitable places. Accommodation is provided for twenty-two cars.

An engine test-bench in a suitable enclosure has been built near one hangar.

Sites have been allocated for oil companies' installations with water and road frontage. Petrol and oil supplies are brought by barge to these installations. Distribution to aircraft is by means of tank lorry. Underground petrol tanks on the aerodrome are not allowed.

A small isolation hospital has been provided giving accommodation for any suspected cases of infectious disease that may be brought in by aircraft from other countries.

Two 23-foot launches built by the Power Boat Company with a cruising speed of 18 knots are provided for control of the alighting area, seaplane channel, etc. They are fitted with a searchlight, signal lamps, crash gear, fire-fighting equipment, six flare-buoys for night landings, and a low-power radio-telephony set for communication with the control tower.

Four 55-inch rubber-covered aircraft mooring-buoys (designed by Short Brothers) are laid in two trots of two. Each trot includes two 17-cwt. single arm and two 20-cwt. clump-anchors, and a 2-ton concrete sinker. A swinging circle of 225 feet is allowed. In addition, two steel drum-buoys of local manufacture with rubber fenders are provided.

On the east side of the landing ground is a completely equipped meteorological station, sited so as to be as far as possible from local influences. It is connected to the meteorological office in the administration building by direct telephone.

Across the centre of the landing ground is a broken line in concrete blocks painted white and flush with the grass-surface. This continues for 300 yards to the east beyond the perimeter of the 1,000-yard diameter circle, the perimeter ditch being bridged for a length of 300 feet at this point. This is intended as an overrun for the take-off of an exceptionally heavy machine if necessary, and for guidance in times of poor visibility and calm winds. There are the usual markings, consisting of a 100-foot-diameter circle and the name "Singapore" in standard-size letters; red and white paint in bands and squares is used to show up curbs, floodlight huts, etc.

Costs.

The total costs of the various works are set out in Table I. The dollar corresponds to 2*s*. 4*d*. in English money.

Conclusion and Acknowledgements.

The airport was opened for traffic on the 12th June, 1937, by His Excellency Sir Shenton Thomas, G.C.M.G., O.B.E., Governor and Commander-in-Chief, Straits Settlements. The opening ceremony was attended by about seventy-five aircraft of various nationalities and by about seventy thousand spectators. Up to the end of 1937, there had been four hundred

TABLE I.—STATEMENT OF COSTS AS AT THE 31ST DECEMBER, 1937.

Item.	Head of expenditure.	Cost: dollars.
1	Excavation, transport, and filling	3,110,404
2	Railway plant	630,000
3	Coolie lines for 1,000 men	144,376
4	Reclamation (Fort road)	549,063
5	Dredging basin	622,541
6	Dredging Geylang river	220,000
7	Drainage	162,447
8	Pitching, etc., to slopes	72,417
9	Fencing	26,505
10	Steel sheet-piling	14,000
11	Hangars	547,904
12	Approach roads	98,916
13	Aprons around hangars	162,797
14	Wharf	35,000
15	Electric wharf crane	25,336
16	Seaplane slipway	209,601
17	Office, including fittings, etc.	380,149
18	Wayleaves	50,628
19	Seaplane shed	62,940
20	Lighting and equipment	147,362
21	Ancillary buildings	38,333
22	Fort road permanent drainage	3,675
23	Contingencies	450
24	Expenses for opening ceremony	5,673
		7,320,517 (£854,060)

and fifteen scheduled arrivals and departures of aircraft, apart from the local operations of the Flying Club, private owners, etc.

The work was designed and executed by the Public Works Department, all reclamation and dredging work being carried out departmentally. Building construction was for the most part undertaken by local building contractors. Mr. F. D. Ward, Government Architect, was responsible for architectural design, and Capt. G. H. N. Reay, Electrical Engineer, was in charge of the electrical works. To these, and to the many civil engineers also of the Public Works Department who have been engaged in various capacities on these works during the 6 years of construction, sincere acknowledgements are due for their co-operation and assistance. The Author is Director of Public Works and Civil Aviation.

The Paper is accompanied by thirteen sheets of drawings, four Tables of costs, and thirty-two photographs, from some of which Plate 1, the Figures in the text, Table I, and the two half-tone page-plates have been prepared.

Discussion.

The President remarked that he would like to say a few words on the Paper, as he had visited the aerodrome during various stages of its construction. The feature of the greatest interest was the way in which the mud was dried out so as to enable material to be deposited on the top of it. It seemed to be an almost impossible task when he first visited the aerodrome-site, since the bund system had only just been started; he had been asked on that occasion whether or not he could suggest any other method of drying out the mud, but he could not think of any other method: it was the only possible way in which it could be done.

The water-mains had to be brought from the Johore reservoirs across a creek not far from the site of the airport. The two 36-inch diameter mains were connected together to form an arch, and were built as a footbridge. The bridge had had to be built on a different site from that originally intended, and no proper explorations had been made. It was hoped that the material would cause a skin-friction of about 1 cwt. per square foot to develop, and with that in view the abutments were designed as hollow boxes turned upside down to give the largest possible surface. The abutments sank gradually, however, and it did not look as though they would ever stop. Fortunately, it was found that by driving some very long piles it was possible to get down to more solid strata.

There was very little in the technical details, apart from the drainage problem, with which he was competent to deal, but he would like to say that natural conditions were not very helpful in that neighbourhood. There was an average rainfall of about 100 inches a year, and it was never possible to be certain when it was going to rain. He had taken out the average rainfall of each month of the year for a good many years and had found very little difference between the figures; there was never a long dry period of sunshine when the surface of the bunded area could dry properly.

Sir Leopold Savile, Vice-President, said that several years ago, when he was in Singapore for the selection of the site for the Singapore naval base, he had been asked by the Air Ministry to collaborate with Group-Captain F. E. T. Hewlett, of the Royal Air Force, in the selection of a site for an aerodrome and seaplane base. Although the site where the Seletar aerodrome was constructed was only 15 miles away from the Singapore airport constructed by the Author, the conditions to be dealt with were entirely different. Sir Leopold then showed five lantern-slides illustrating the site of Seletar aerodrome, which consisted largely of creeks and jungle, the country being therefore difficult to survey. When the Johore cause-

way was made it was very difficult to find any granite suitable for the quoins, the only satisfactory material being obtained from boulders on the edge of one of the creeks shown in the lantern-slides.

Turning to the Paper, he thought that every one would realize that the Author had had quite a different proposition to deal with from that of the Royal Air Force at Seletar; there was not much doubt that, in spite of the fact that at Seletar the country was very bad, the Author had had a far more difficult problem, and one which, considering its magnitude, had hardly ever been solved before. To take a site all of which was under water at high tide and a large proportion of which was under water at low tide, with mud over an indefinite depth, and to construct a stable landing ground on it, was a work the undertaking of which required an enormous amount of courage.

A point to notice was that the whole work had been carried out departmentally without going to contract, and in Sir Leopold's view there was not the slightest doubt that in the case in question that was the right procedure. The engineers of Public Works Departments in the East were used to carrying out big works themselves in a very cheap way. There was very little doubt that if an attempt had been made to let a contract, it would have been very difficult to have tied down a contract to allow for the possible alterations which might have had to be made during the construction of an aerodrome of the kind in question; in any case, a contract would have been liable to lead to fairly heavy claims before the work was completed.

In regard to the suggestion that the work might have been done more cheaply if it had gone to contract, it would be noticed that on p. 73 the cost of excavation and transport of the dry material, of carrying it more than 3 miles by railway, and of depositing it in the reclamation, worked out at about 8*d*. per cubic yard, and he could not imagine many contractors tendering on that basis. He doubted whether, even if they had tendered on that basis, they themselves could have carried out the work as cheaply as the Author had been able to do.

The reinforced-concrete buoys (*Fig. 9*, p. 79) were 8 feet 6 inches in diameter, but the sides were only 4 inches thick. In the experiments which were carried out on the deterioration of structures in sea-water, it was found that any cover for concrete in sea-water under 2 inches was liable to deterioration within a comparatively short time, and with a thickness of 4 inches it was quite impossible to get 2 inches of cover. Singapore, moreover, had a particularly bad climate for ferro-concrete, and a box of the kind in question in that climate would have a very damp hot atmosphere inside it which would tend to make the inside deteriorate, whilst the outside would always be subject to sea air. He would be inclined to doubt whether a ferro-concrete buoy was the best type for the purpose, but no doubt the Author had very good reasons for using it, and perhaps in his reply he would explain his reasons for doing so.

Mr. C. L. Howard Humphreys observed that in the section of the Paper headed "General Considerations" (p. 71), the quantity of filling which it was estimated would be required was stated to be 8,770,000 cubic yards, and he would like to know whether the Author could say how much was actually used. Experience in the Zuyder Zee, for example, had showed that where there was a soft bottom it was quite possible to exceed the estimated quantity of filling by 40 per cent., and in Valparaiso harbour there was a record that in certain parts the estimate was exceeded by 57 per cent.

Another point on which further information would be of interest occurred on p. 72, where the Author classified the various soils which he found in his borrow pit, and noted that class C, the lowest material, was very good. The Author did not mention class B, and, as they were both clays with varying percentages of sand and both had to be drained, it would be interesting to know the Author's opinion of class B soil.

The President had referred to the dewatering of the mud. When the Author had constructed his bunds he had had to dewater the mud, and it was interesting to find it stated (p. 74) that borings showed that penetration of the tipped soil had reached a depth of more than 40 feet in places when it was being tipped on to the soft mud before it was dewatered. The Author also said that after the mud had dried a depth of 12 feet of filling depressed the mud only approximately 2 feet. To what depth had the mud to be dried in order to give it such an enormous increase in bearing capacity?

The aerodrome was in the form of a dome, and had a gradient of 1 in 190. What was the maximum gradient on which it was possible to land an ordinary commercial machine without risk and inconvenience to the passengers? With a big aerodrome of that kind there was the risk of two machines simultaneously approaching the centre from the opposite ends of a diameter, and colliding because the pilots could not see each other.

The piling also presented a small point of interest. It was stated that the foundations for the hangars were on piles varying from 55 to 90 feet in length. Such piles were very long, and he wondered whether the Author had found, as Mr. Humphreys had found, that a pile driven into soft ground would, if left for a short while, become so gripped by the ground as to become completely immovable, after being what had previously been regarded as apparently hopeless for bearing purposes.

The administration building was a very fine piece of work; too often, however, when landing from a ship or from the air on British territory, passengers were taken into a wooden or corrugated-iron shed, and made rude mental comparisons with arrangements in other countries. He was also very glad to see that the Singapore airport, apart from the buildings, was wholly the work of an engineer.

There was one thing mentioned in the Paper which was a ground for hearty congratulations to the Author, and he was certain that it was a

ground of great satisfaction to the government of the Straits Settlements; it was a feat which he was sure had never been accomplished before in the annals of The Institution: it would be seen from the figures of cost (p. 90) that what the Author spent on contingencies amounted to only 0·00006 per cent. of the total cost!

Mr. Malcolm Millar said that he wished to give some idea of the results of the work described in the Paper from the point of view of the air operator. At any point where he might reasonably expect a fair volume of traffic, an air operator's requirements were manifold, but, broadly speaking, they could be grouped under the following heads. First and foremost came safety; there was no room for risk in present-day air transport. Secondly came aids to navigation and approach, thirdly general facilities, and fourthly accessibility.

The problem under the first heading, that of safety, was easy to understand, as it was so obviously desirable to have both aerodromes and seadromes free from surrounding hills and clear of obstruction on all sides, having an adequate area reserved for the purpose in view, and, so far as the landing ground itself was concerned, a surface firm enough to avoid the risk of bogging, even in the worst conditions. There were many aerodromes in use to-day where those obstacles and difficulties did exist, and, although they did not constitute any real danger, they added to the many difficulties with which an air pilot had to contend; it was the constant task of the air operator to reduce those difficulties to a minimum.

Aids to navigation could be divided into so many different sub-headings that it would be necessary for him to confine himself to the broader principles. Aids to navigation were of prime importance in the successful operation of present-day air transport, which should, roughly speaking, be able to carry on its work under any conditions. Much still remained to be done in that direction. Further, of those aids the two most important went together: wireless and meteorological information. With the former there were direction-finding stations and blind-approach schemes, and following them there were night and blind-landing equipment, obstruction lights, marker beacons, and similar equipment.

Under the third heading, general facilities, could be grouped all those other things which appertained to airport equipment. From the point of view of the passengers the air operator tried to give them shipboard comfort and railway service at airway speed. The air line wished to take its passengers from one point to another with the least inconvenience to them. From the air operator's point of view the operator desired to have all his equipment so placed that he could carry on his own work as efficiently and rapidly as possible. Time was money, and in actual operations a period of 1 hour could also mean a distance of 160 miles.

Lastly, there was the question of accessibility and proximity to the point which the aerodrome served. It was somewhat absurd to spend

$2\frac{1}{2}$ hours on a journey of 400 miles and perhaps $\frac{3}{4}$ hour in reaching the aerodrome at each end, but that position did exist in many places.

It was of interest to see how the Singapore civil airport met those requirements. From the point of view of safety, the landing ground left little to be desired. There were easy approaches from all sides, and the hangars and administration building were so placed as to offer little or no obstruction for a landing or take-off in any direction. The landing ground was amply large enough for present-day commitments, and although it might be difficult, an extension of the area or the provision of runways should not be impossible if it were ever found necessary. The actual marine alighting area was in the Singapore harbour proper. As in the case of a landing ground reserved for land craft, so would those operating flying boats like to see an area entirely reserved for their craft, and so placed that the moorings and administration buildings were situated on the boundaries of that area. With the approach channel as it was at present, in certain conditions flying boats might be left with rather too long a distance to taxi to their moorings, both from the engineering point of view and for the comfort of the passengers, but it had to be remembered that flying boats were, relatively speaking, still in their infancy, and the present arrangements at Singapore were in advance of those at many other places in the Empire.

With regard to aids to navigation, Singapore was splendidly equipped, as was Malaya generally. The existing organization included a civil aviation wireless station, which kept in touch with all craft approaching or leaving Singapore either until they had arrived or until they had safely reached the next point ahead. There was a medium-wave direction-finding station and a meteorological office in the administration building. As was stated in the Paper, visibility could be reduced to nil during severe, but fortunately usually localized, storms, which were generally of short duration; at all other times visibility was good, fog being unknown, and there was no need for blind-approach systems such as were fairly widely used in America. The night lighting for the landing ground was excellent, and the buildings, which might constitute a danger to craft making a particularly low approach, were adequately lit. The illuminated wind-tee enabled pilots to ascertain the wind direction without any difficulty.

For the marine area it had been found that the old type of paraffin flares mounted on twin hydroplane floats still provided the most suitable means for night landing. They had disadvantages, and it was not possible, as with the aerodrome floodlights, immediately to illuminate the alighting area at a moment's notice. The approach was large enough to allow two flying boats of the Empire type to manœuvre and pass each other without difficulty.

Under the heading of "facilities", the general facilities were good, and those for land planes left nothing to be desired. The telescopic covered ways provided enabled passengers to be embarked and disembarked in

complete comfort during the heaviest tropical storms. The hangars were conveniently situated, and the system of doors enabled large transport craft to be housed with the least possible difficulty.

With regard to marine craft, operators would naturally prefer to see arrangements such as a dock or a railway station, or at least facilities comparable with those enjoyed by the land craft, but there again Singapore was some way in advance of other marine stations, being able, as it was, to handle passengers and loads effectively in adverse conditions. Viewed from the ideal arrangement, the customs and emigration departments, at present housed in the administration building, were perhaps a little too far off, even though the journey was undertaken by car. It was desirable in marine air transport to have some arrangement for the berthing of aircraft which would allow passengers to be disembarked direct on to a covered pontoon or platform, whence they could continue under cover to comply with the various official formalities. However, possible alterations and alternatives in that direction were already being actively considered by the Civil Aviation Department of the Government.

It was always preferable to have the hangars as near as possible to the slipway to prevent loss of time. Even with a skilled shore crew, towing craft to the hangar sometimes took longer than beaching them. It was not always possible to beach them, however, and it would be of immense help if the necessary maintenance could be carried out under cover when the flying boats were afloat. Other excellent facilities for marine craft were the arrangements for lighting buoys at night, the mooring buoys, the crane, which had already been used for handling engines, and the ability to beach craft at any state of the tide. Singapore was probably more fortunately placed in regard to accessibility than any city of comparable size. From the airport proper to the main hotels was not more than 5 minutes by car, and it was only 2 or 3 minutes farther to the business area.

To sum up, Singapore civil landing ground met almost every possible need of the most exacting air operator. When it was borne in mind that the first flying boats did not operate regularly to Malaya until 8 months after the opening of the airport, the excellence of the marine facilities was, he thought, ample proof of the energy and foresight with which the problems had been tackled by the Government, and particularly by the department concerned. From what he had seen in some fourteen different countries, he would like to support the view which had been expressed in other quarters that Singapore was the Empire's finest civil airport.

Mr. G. B. Gifford Hull remarked that, like Sir Leopold Savile, the costs of the work had impressed him. It appeared that transport over a distance of 3·38 miles was done at a cost of something like 2*d.* per cubic yard. He would be grateful if the Author could amplify the figures given. The cost for excavation, transport, reclamation, and staff and general, came to 28·71¢ per cubic yard, whereas the same items in Table I (p. 91)

gave a figure of 44¢ per cubic yard. He noticed that the cost of railway plant was shown as $630,000, but that did not seem to be included in the costs at all. Before he very reluctantly dispelled an illusion which he had had for some years—namely, that when he and his associates had been building the earth dams for the President's water undertaking at Singapore they had done their work economically—he would be very glad if the Author could give some greater details with regard to his figures. He noticed that the costs applied to cubic yards loose in the wagon, which would in any case put them up to some extent.

An interesting item in Table I was the coolie lines for 1,000 men, the cost of which were shown as $144,376. A figure of $144,000 for 1,000 men worked out at $144, or 360*s*., per man. If the regulations governing the number of cubic feet of space per man which obtained in the East—namely, 350 cubic feet—applied in the case in question, the cost worked out at over 1*s*. per cubic foot, for which proper houses could be built even in Great Britain, where labour was expensive. He would like to know, therefore, what kind of coolie lines were provided, whether or not they were of the ordinary type usually built in Singapore with wood, whether or not they had water-borne sanitation, and whether or not the cost of paths round them, and so on, was included, because coolie lines for the 3,000 men employed on the President's recently finished work in Hong Kong, and which were regarded as excellent coolie lines for the East, cost less than $144,000, although for three times the number of men.

Mr. C. E. O. Wood said that he had been representative of the Works Directorate of the Air Ministry in the Far East for 10½ years, and thus had been most interested in the remarks of Sir Leopold Savile, who went out there in the early days to select the site for the future Royal Air Force station at Seletar. It had been his pleasure to have been connected with the Author during the period of construction of the civil airport in Singapore, which from an earthwork point of view was approximately 7 times the size of the Royal Air Force station, and he entirely agreed with Sir Leopold Savile that the Author's problems and those of his resident engineer were much more difficult, from an earth-formation viewpoint, than were the problems at Seletar. Seletar was an undulating site which involved, from the point of view of the landing ground alone, earthworks to the extent of 1,000,000 cubic yards; during its construction they had realized that if 1 cubic yard of earth were excavated and removed to a new site, it did not occupy 1 cubic yard in its new position owing to the fact that, according to the formation on which it was being deposited, it was necessary to contend with the shrinkage- and compression-factors; at Tengah aerodrome the losses were 18½ per cent through the compression- and shrinkage-factors. That was taken into account when dealing with Sembawang, where 3,500,000 cubic yards had to be dealt with, and he estimated there that possibly the final figure

for loss would be in the neighbourhood of 9 per cent., owing to the increased volume.

It had been his privilege to be called in by the former Governor of the Straits Settlements, Sir Cecil Clementi, to examine with him the proposed site for the civil aerodrome of Singapore in February, 1930; at that time it was proposed to site the landing ground at Tengah, which was now the Royal Air Force Station, but Tengah would not have had any marine facilities, and after careful inspection of the site Sir Cecil Clementi decided that a site had to be selected within 5 miles of Government House, which would provide requirements not only for land machines but also for seaplanes and flying boats. Within 3 days of Sir Cecil Clementi's visit the site which had been developed was selected, and he was sure that the Author and all others concerned would agree that Sir Cecil showed marked foresight in emphasizing that the Kallang basin was the ideal site for the future civil airport of Singapore.

Mr. C. I. Robinson, referring to the doming of the aerodrome, said that a rise of 8 feet at the centre over the circumference sounded a good deal, but he believed that he was correct in stating that despite that tremendous amount of doming, aircraft had experienced difficulty with regard to water on the surface when landing or taking off. The great amount of subsoil drainage was referred to in the Paper, and it seemed difficult to realize that at any time there should be so much water on the surface that the wings of aircraft in motion were damaged. It might be that the present vogue for low-wing machines would pass and that that danger would pass with it, but it would be of interest if the Author would say whether or not there was any way of eliminating the danger of damage to aircraft from surface water.

He noticed that 16-inch octagonal piles were used in connexion with the slipway, and were apparently surrounded by 21-inch spun-concrete pipes. What was the reason for putting pipes around piles of that size?

At first sight it would appear that dredged material should have been partially or entirely used for filling the swamp. The Author gave as his reason for not doing so that they had some other use for the dredged material; eventually, however, nearly 1,500,000 cubic yards of material were dumped to sea. That, it would appear, might well have been used in the initial stages on the site. The Author stated that dredgings would not have proved very satisfactory material, but since the area had had to be bunded in cells and each cell drained in order to provide a surface upon which material could be dumped, Mr. Robinson could not see that there would have been any disadvantage in pumping dredged material into those cells before they were drained off.

Mr. H. G. Lloyd remarked that in view of the fact that the construction was carried out in a country where there was a high rainfall and also where there was a moist atmosphere, it would be of interest if a few details could be added to the Paper which would indicate the proportions of the concrete

which had been used in the various works, and also the steelwork which had been used in the reinforcement. Another point which was of considerable interest, and which had already been touched upon by Sir Leopold Savile, was how much cover there was to any reinforcing steelwork, especially in view of the fact that in some places the concrete was only 4 inches thick. It would be of interest also to know whether the materials used for the concrete work were obtained on the site, and what those materials were, together with some indication of their sizes, so that later on, when reference was made to the Paper to find out what had happened, information would be available regarding the materials used in the construction. Some reference might also be made to the cement.

Mr. R. F. Lloyd-Jones, referring to the drainage of the aerodrome, said that with the rainfall common in Great Britain it had become very difficult to ensure that surface water was kept off the landing surface; with a grass-surfaced aerodrome the only way in which it had been found practicable to achieve that end was to put a fairly stiff gradient on the surface, or else to fill the drains and drain trenches with porous filling up to the surface to intercept the surface water. In the Paper it was not stated whether or not the drains were filled to the surface, or to a level anywhere near the surface, with porous material. It was usually found easier to do that because, where there was a very large area of water, pooling would almost certainly occur in places if reliance were placed on the gradient of the surface to remove the water.

He was not quite clear what the bearing capacity of the ground was at the airport, and whether it was well able to resist the wear of aircraft. There were many reasons for supposing that runways would be necessary at almost every major airport, and he wondered whether that point had been considered in the design of the airport at Singapore. He noticed that the drainage was set out on a purely symmetrical plan, and did not appear to be designed to take the surface water from any impervious areas which might be put down in future.

Had the question of having a V.H.E. direction-beacon been considered? Despite the fact that there was no fog at Singapore, visibility was apparently reduced to zero at times; in any case, a beacon of that kind was useful for approaches through low cloud, although he did not know whether low clouds were experienced at Singapore.

The Author, in reply, observed that Sir Leopold Savile was interested in the question of the reinforced-concrete buoys, and that the same point had been referred to by other speakers. The first reason for utilizing reinforced concrete in preference to steel was that of cost, since steel buoys for the same duty would have cost more than twice as much as reinforced-concrete buoys. A good deal of experience had been gained with reinforced-concrete pontoons and other marine structures at Singapore, and it was felt that it was justifiable to try reinforced concrete for the buoys under discussion. He admitted that 4 inches appeared to be

rather thin, and did not give a great deal of cover; so far as he was aware, however, the buoys were satisfactory, and none had so far shown signs of deterioration, although he admitted that experience was limited to 2 years.

Mr. C. L. Howard Humphreys had referred first to the actual quantity of filling. He was sorry that he could not give that figure without referring to his office in Singapore, but his impression was that the difference between the estimated and actual quantities was not very great. They had been very fortunate in some respects; the amount of depression of mud surface due to the superimposed weight was rather less than they had expected, and that had helped to make up for the other places where bunds had slumped. Mr. Howard Humphreys also referred to the classes of soil mentioned in the Paper. The Author had expressed a preference for what he termed class C soil largely on account of its greater porosity. He thought that better conditions would have been obtained in the end had all the material been of that category and had it not been necessary to utilize first of all the class B soil, which had only 24 per cent. of sand instead of 43 per cent. Class B soil had had to be taken as well as class C, however, because class B was on the top. Another question was the depth to which the mud was dried out. The herring-bone drains had been dug to a depth of 2 or 3 feet, and they had endeavoured to keep the area pumped out until such time as they judged that it would take the weight of the material. It was largely a matter of trial and error, as was most of the job. They found by a certain amount of experience that when it was possible to walk about comfortably on the surface, it was necessary to wait another week if the weather were fine before commencing dumping.

Mr. Howard Humphreys had also asked a question about the maximum gradient which could be permitted on a landing ground from the point of view of the pilot. In certain circumstances it was possible to land on a very steep piece of ground indeed, but for a landing ground which had to be in constant and safe use by civil air traffic, he did not think the gradient should exceed 1 in 50, and it was better to have it not so steep as that. It was possible, especially with a grass surface, for an aeroplane to have its wheels locked when it came in to land. The pilot put on the brake to pull the machine up, and if the grass were wet the wheels might lock and the machine would skid over the surface with no signs of slowing down. That might happen to some extent on a level surface, and with a gradient steeper than 1 in 50 it might be impossible to stop the machine.

In Singapore they were used to long piles. Under the Drill hall at Singapore piles as much as 125 feet in length had been used, and a length of from 70 to 100 feet was quite common. It was true that a pre-cast pile might "seal itself up" with skin friction while an extension was being cast, but it was necessary to proceed with the extension, because the pile might not have sealed itself up; one or two piles had occasionally been

left in the hope that extension would not be necessary, but when a test load had been applied the pile had run away. One or two piles had been broken in carrying out the work, particularly down at the jetty, but they had managed to get over that difficulty.

Mr. Millar had spoken about the provision of a kind of railway station into which the flying boat would taxi at the end of the journey, and in which all the repairs could be done. The Author remarked that the flying boat was a fairly recent development and he did not know that it was a permanent one. He was inclined to think that it was, and that the large aircraft of the future would be flying boats. That being the case, it would be necessary to tackle the problem, which he did not think had been attacked anywhere yet, of marine terminal stations for flying boats. One of the difficulties met with was the different types of flying boat: some had wing-tip floats and others had short stubs to prevent them turning over; those and other differences in size and design made it extremely difficult to design a terminal station which would suit all present and future types, and he did not know at the moment what the answer to the problem was. It was, however, a matter which would have to be dealt with by collaboration between airport authorities and aircraft manufacturers.

Mr. Hull had raised some questions about the earthwork costs. The Author regretted that Mr. Hull might have been misled by the figures given on p. 73. What had happened was that in submitting the Paper he had enclosed a detailed Table showing all the costs in one typical month, when the work was in full swing. Unfortunately the Table had not been reproduced with the Paper, and only the summarized figures had been extracted therefrom. It would be appreciated that the first item in Table I (p. 91) included a great many preliminary and other expenses of a non-recurrent character. The costs on p. 73 were purely operating costs and did not include costs of plant and machinery; he could, if necessary, furnish the detailed costs in question. The item in Table I "Coolie lines for 1,000 men: $144,376" should be amended to include the words "and quarters for supervising staff." Mr. Hull's figure of $48 per man was about right for that class of coolie line.

Mr. Robinson and Mr. Lloyd-Jones referred to water lying on the ground. It was perfectly true that it was not possible to have a grass surface, however smooth, absolutely dry just after a heavy storm, unless the ground was extremely porous. Possibly chalk might remain dry, but he had not had enough experience of chalk recently to be able to say whether chalk was sufficiently porous. It was inevitable that grass itself should hold up a certain amount of water, and he had made a number of measurements at times of the amount of water on a typical piece of the surface, and it was something like 1 inch in depth. He had also seen aircraft landing on concrete runways after very heavy rain, and, just as in the case of a grass surface, it created the impression of a destroyer

going very fast. He did not know of any case of damage to an aircraft owing to the water on the surface. The presence of water caused drag which would pull up a landing aeroplane more quickly and would likewise absorb some of the power when an aeroplane was taking off. Nevertheless there had been only one case of a deferred departure since the airport was opened in June 1937. That had happened in January of the present year, when the captain of a Dutch aircraft decided, without looking at the ground to see if he could take off safely, that the weather was too bad for him to proceed. As far as he knew, there had been no case of a deferred arrival on account of airport conditions. The machines did at times cut the grass up. When landing after heavy rain, with the grass still very wet, the wheels of an aircraft sometimes locked when the brakes were applied, and tore the grass up. All that it was necessary to do in such a case was to plant a little more grass. There was, of course, a big area of concrete near the building itself where most of the manœuvring was done. On the whole, the grass surface was standing up very well. There were one or two places where recent landing of heavy aircraft had left a mark, but there was no danger to the aircraft of sinking or anything of that kind; the surface was much too hard. As a matter of fact, it was impossible to drive a peg into the ground.

Mr. Robinson referred to the pipes round the piles of the jetty. They were simply put there for additional protection against bumping by boats.

With regard to dumping the dredgings at sea, the main reason for that was that the dredgers were not available for the seaplane channel until the airport reclamation was well in hand, and that the output of the dredgers then exceeded the rate at which the material could be accepted at the alternative reclamation. It was most important to complete the airport as quickly as possible, and to have used some of those dredgings for the airport reclamation would have materially delayed the work. The key to progress was the rate at which the bunds could be laid down, and had there then been delay in filling that area partially with dredgings, he was afraid that the completion of the airport might have been quite considerably delayed.

Mr. Lloyd inquired about the concrete materials. Excellent granite and sand were available on Singapore island, whilst cement and steel were imported from England. To a very large extent, a mix of 1 : 2 : 4 was used in the concrete work at the airport, the granite being crushed to ¾-inch gauge for reinforced concrete. When possible 2 inches cover was given, but as Mr. Lloyd and other speakers had observed, it was not practicable to give that in all cases, and then 1 inch had to suffice.

Mr. Lloyd-Jones had referred to drainage from possible future runways. The Author did not consider that drainage from any future runways which might be put down would present material difficulty. The curbside drains could deliver at convenient intervals down into existing intercepting drains; he had not yet worked the design out in detail. It was

possible that it might become necessary to put runways down, but a full system of concrete runways at Singapore would cost something like $2,000,000, and for every year for which the putting of them down was deferred there was a saving, at 3 per cent., of quite a considerable sum of money.

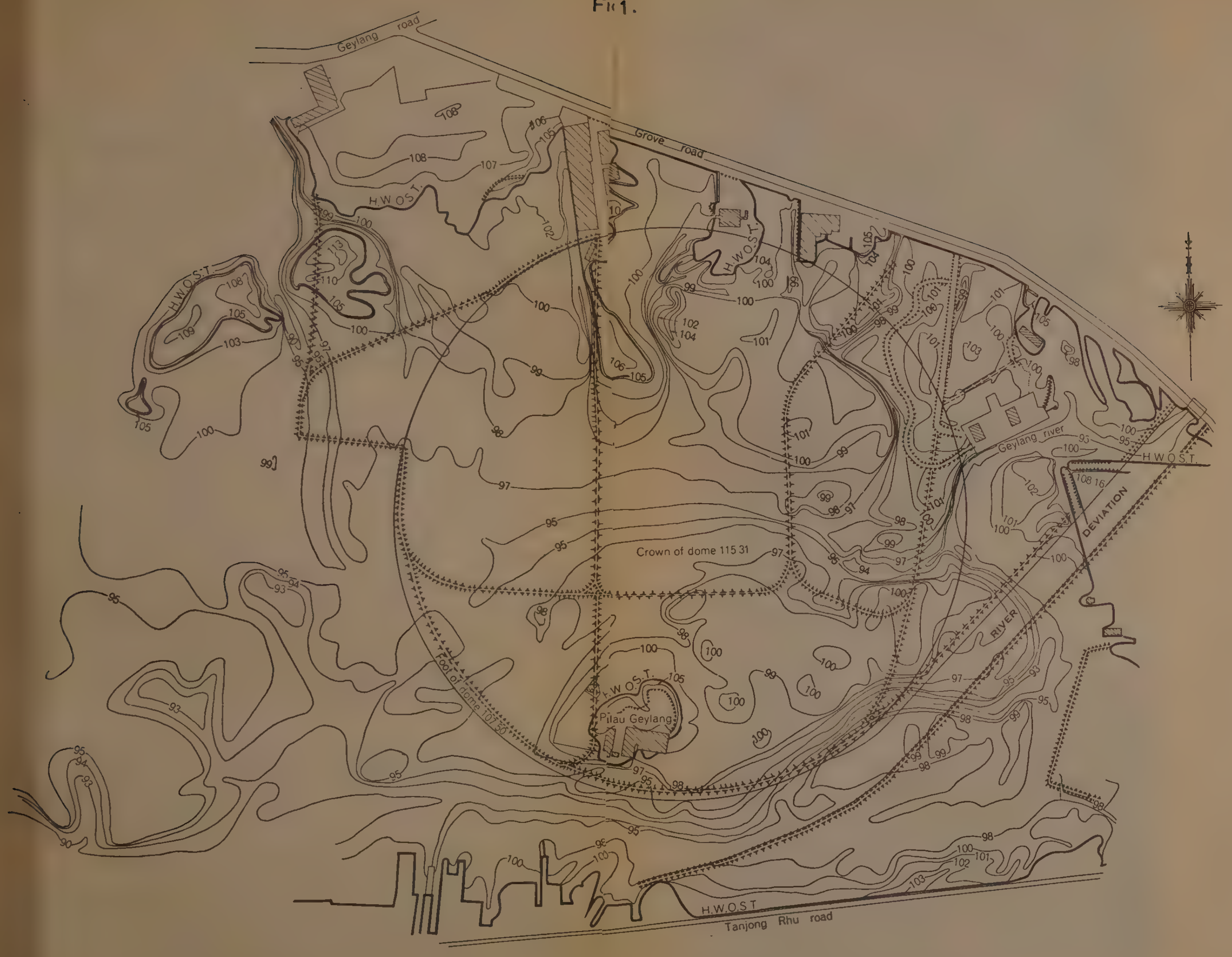
The test for bearing capacity of the landing-ground surface was 3 tons per square foot, the tests being carried out by means of loaded lorries.

No need had yet been shown for the provision of any wireless approach systems; when visibility was bad, the air was generally so disturbed as to make an approach and landing inadvisable, but the duration of such storms was, fortunately, brief.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October, 1939.—SEC. INST. C.E.

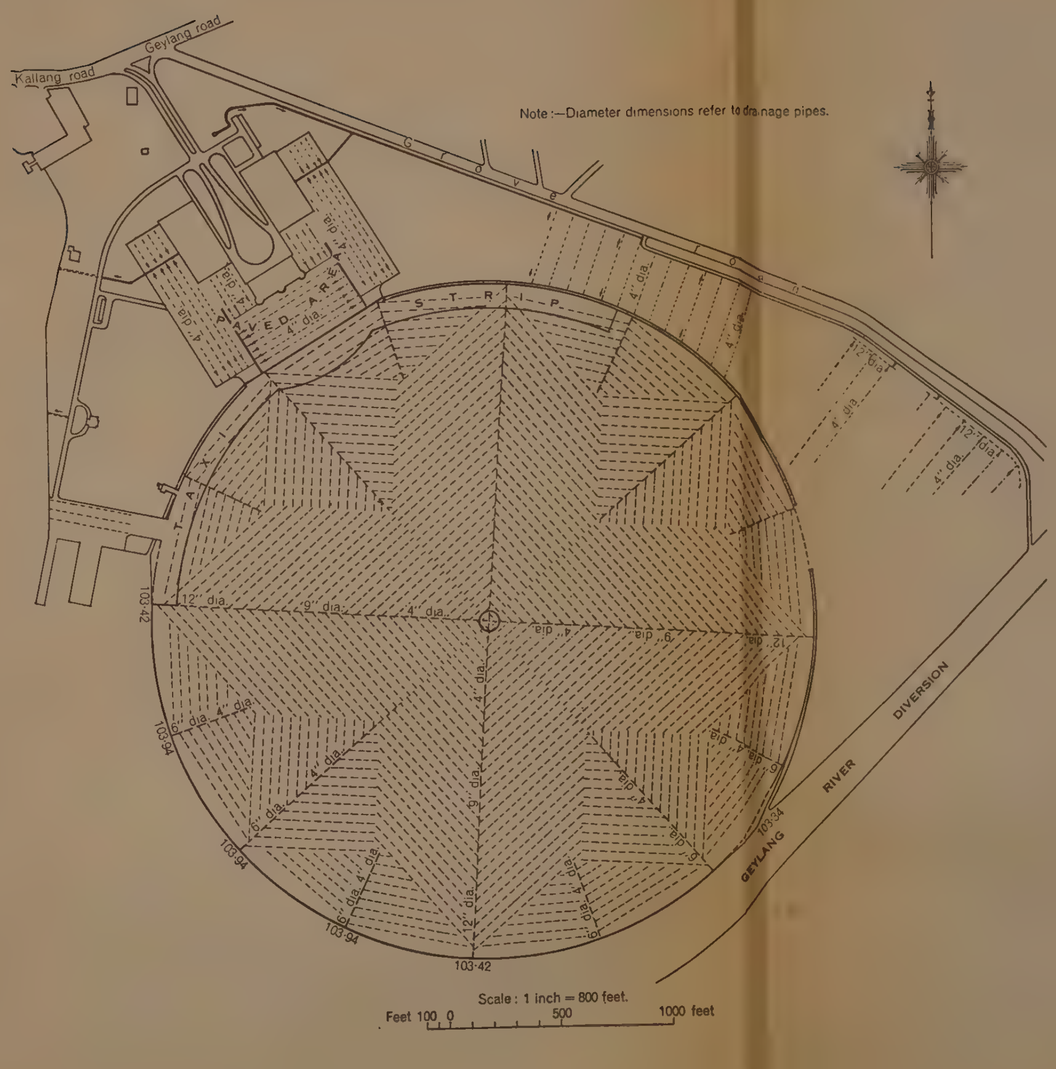
THE SINGAPORE AIRPORT.

FIG. 1.



CONTOUR PLAN.

FIG. 5.



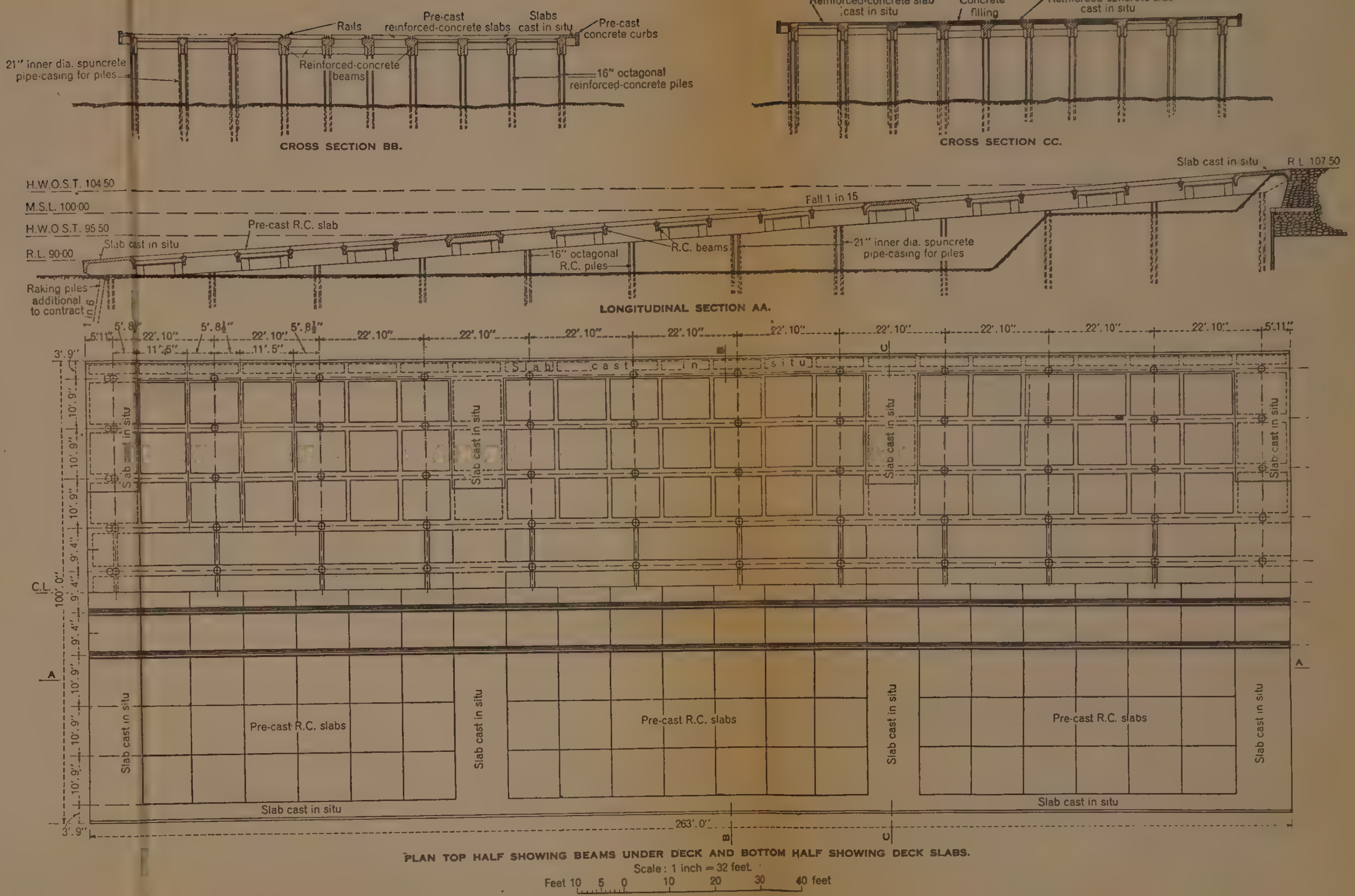
SUBSOIL DRAINAGE SCHEME.

FIG: 10.

LAYOUT OF BUILDINGS AND HANGARS.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

FIGS: 17.



PROPOSED SLIPWAY.

R. L. NUNN.

EXTRA MEETING.

2 May, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

PRESENTATION OF THE JAMES ALFRED EWING MEDAL.

The President remarked that his first and very pleasant duty was to present the James Alfred Ewing Medal to Professor A. H. Gibson, D.Sc., M. Inst. C.E. Sir Alfred Ewing, who was an Honorary Member of The Institution, had died in 1935, and in 1936 Lady Ewing, certain members of The Institution, and Sir Alfred's personal friends, had founded an endowment fund for the annual presentation of the James Alfred Ewing Medal to a person, whether a Member of The Institution or not, for especially meritorious contributions to the science of engineering in the field of research.

The procedure was to invite the Presidents of the Institutions of Mechanical Engineers, Electrical Engineers, and Naval Architects, to submit names for consideration. The President of the Royal Society and the President of the Institution of Civil Engineers then selected the name they considered most worthy for the presentation. It was a great honour to be the recipient of the Medal. Sir William Bragg, the President of the Royal Society, and Mr. Binnie had given very careful consideration to the names put forward, and they had decided that Professor Gibson's work had been of the very greatest value to civil engineers, particularly in the direction of model-experiments. He had also been good enough to contribute no less than eleven Papers to the Institution of Civil Engineers.

As Professor Gibson was so very well known to the members it was not necessary to take up more time, and he would therefore ask him to receive the Medal.

Professor A. H. Gibson said that he appreciated more highly than he could say the honour conferred upon him by the award of the James Alfred Ewing Medal. He could think of no form of recognition of the work of his colleagues and himself which would give him greater pleasure.

THE JAMES FORREST LECTURE, 1939.

The President said that before he called on Dr. Karl von Terzaghi to deliver the forty-fifth James Forrest Lecture he wanted to make a few introductory remarks, although he felt they were not really necessary, because Dr. von Terzaghi's work in soil mechanics had gained him a unique position in civil engineering in all countries. He had been a lecturer in

Turkey, Austria, Germany, and the United States of America. His first book on the subject was published in Germany in 1925, and he had devoted the greater part of his time to elucidating problems in connexion with soil research. In Great Britain rather slow progress had been made in soil research, but the value of the work which Dr. von Terzaghi and others had done was now being realized.

"Soil Mechanics—A New Chapter in Engineering Science."

By Karl von Terzaghi, Dr.-Ing., M. Inst. C.E.

Introduction.

Almost half a century has elapsed since Dr. William Anderson opened the series of James Forrest Lectures with a brilliant discourse on "The Interdependence of Abstract Science and Engineering." Through the courtesy of the Secretary of The Institution, I was given an opportunity to read Dr. Anderson's Lecture [1]. After I finished absorbing the substance of that inspiring Lecture, I realized that I could do no better in my own Lecture than to add one more little detail to Dr. Anderson's masterly exposition. This little detail comprises the mechanics of soils.

In Dr. Anderson's day the principal objective of the scientific approach to engineering problems consisted in establishing a reliable stock of knowledge concerning the mechanical properties of our construction materials, such as steel and concrete, and the application of this knowledge to structural engineering. The struggle was by no means easy, and Dr. Anderson did not hesitate to express his opinion on the nature of the resisting forces. He said: "The history of scientific research teems with instances of discoveries which at first seemed to have no practical value, but which nevertheless have ultimately proved to be of the utmost importance to the engineer. Notwithstanding such experience, and even in these days [1893], there are so-called practical men who are utterly intolerant of either time or money being devoted to investigations which seem incapable of bearing immediate fruit."

Since those days structural engineering has emerged from the struggle as an accomplished science, with very little margin left for guesswork and for the so-called "acts of God." As a consequence pioneering activities have moved from the scene of their victories into other regions still uncontaminated by scientific reasoning. One of these regions is located beneath the base of the products of structural engineering. It is the subsoil.

Year after year engineering periodicals report the expensive consequences of the allegedly lawless behaviour of soil. Many more accidents remain concealed in the memories of the authors of unsuccessful designs, because ignorance and vanity are concomitant. Less than 10 years ago

[1] Minutes of Proceedings Inst. C.E., vol. cxiv (1892–93, part IV), p. 255.

the Foundation Committee of a well-known engineering society decided, at one of its meetings, that the word "settlement" should be avoided in public discussions, because it might disturb the peace of mind of those who are to be served by the engineering profession. Even to-day it is sometimes very difficult to learn the truth about foundations or dams which have developed into sources of trouble. These and many other dismal facts concerning the subsoil induced progressive engineers all over the world to turn the searchlights of scientific investigation from the superstructures into the dark spaces located beneath the towering products of structural engineering. The movement started simultaneously in several countries some 25 years ago, and the result of the concerted action was given the name "soil mechanics." Similar to structural mechanics, it deals with both the mechanical properties of the materials involved and with the application of the knowledge of these properties to engineering problems. Officially the christening of the newly-born science took place in 1936, at the First International Conference on Soil Mechanics and Foundation Engineering at Harvard University in Cambridge, Massachusetts.

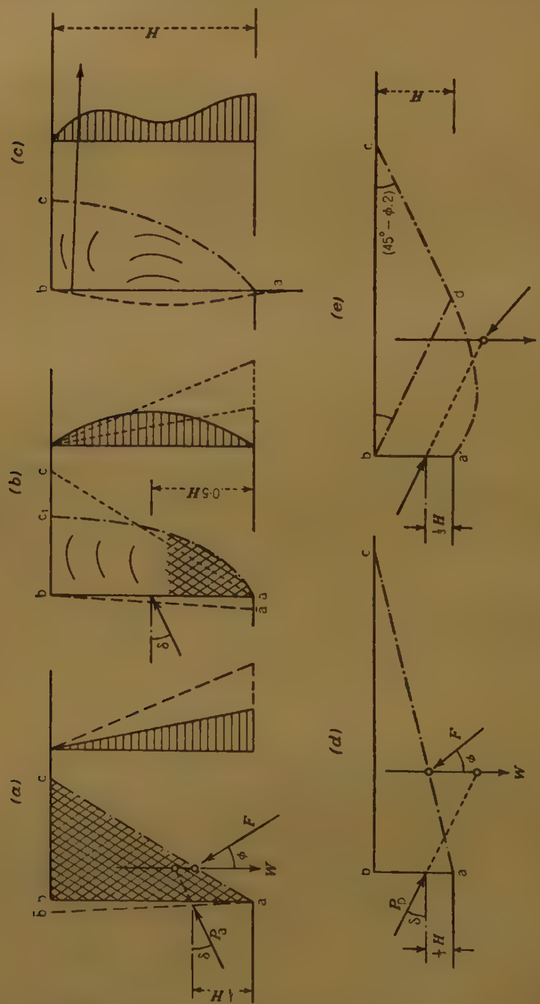
The value of an engineering science is determined by what it can accomplish as a tool in the hands of the practising engineer. Hence in order to judge the value of soil mechanics, let us compare the degree of reliability of the pre-war methods with those of to-day, based on experience blended with a theoretical insight into the mechanics of what happens in the field. The subjects we shall cover are the pressure of earth on lateral supports, the stability of slopes, the safety of dams and weirs with respect to the danger of piping, and the performance of foundations with and without piles. In dealing with each one of these subjects I shall start with a brief review of past practice and its shortcomings, then I shall discuss the causes of these shortcomings, and finally I shall describe the improvements which have been realized during the last decades.

Pressure of Earth on Lateral Supports.

During the last century and up to the present day the pressure of earth on lateral supports, such as retaining walls or the timbering of cuts, has been computed by means of one of two theories, known as Coulomb's theory and Rankine's theory. From experience and from experiment, we know that the conditions for the validity of Coulomb's theory are more often satisfied under practical conditions than are those for the validity of Rankine's theory. The following discussion will therefore be limited to the first of these theories. For the sake of simplicity it will also be limited to backfills with negligible cohesion, such as sands.

Coulomb's theory was published as early as 1773. It is based on the empirical fact that the failure of a laterally supported bank occurs as a result of the shearing resistance of the earth being exceeded along a surface of sliding, such as ac in *Figs. 1*, which starts at the lower edge of the

lateral support and rises towards the surface of the backfill. If the wall yields away from the fill, such as shown in *Figs. 1* (*a*), the average slope of the surface of sliding is greater than 45 degrees, and the corresponding lateral pressure is called the " active pressure " of the earth. If, on the other hand, the lateral support is forced towards the fill the surface of sliding



Figs. 1.

rises at an average angle of less than 45 degrees, as shown in *Figs. 1* (*d*) and (*e*), and the lateral force required for producing this type of failure is called the " passive earth pressure." Coulomb's computation of the earth pressure was based on the following assumptions :—

(1) That the shearing resistance *s* per unit of area, along any section

through the earth, is equal to

$$s = p \tan \phi, \quad \ldots \ldots \ldots \quad (1)$$

where p denotes the normal pressure per unit of area of the section and ϕ the angle of repose of the earth.

(2) That the surface of sliding is perfectly plane.

(3) That along the surface of sliding the shearing resistance of the earth is fully mobilized.

These three assumptions make it possible to compute the total intensity of the lateral pressure, but they do not suffice to determine the location of the centre of pressure. In order to obtain information on this vital point, Coulomb was obliged to supplement his set of assumptions by a fourth:

(4) That in every point of the sliding wedge, abc in *Figs. 1*, the state of stress in the soil corresponds to what is known as a "limiting state of equilibrium." This assumption leads automatically to the following conclusion. If both the back of the lateral support and the surface of the backfill are plane, the state of homogeneous failure in the sliding wedge involves a hydrostatic distribution of the lateral pressure over the back of the support. The corresponding centre of pressure is located at one-third of the height of the wall.

Based on these four assumptions Coulomb derived his well-known formula for the computation of the lateral pressure. Practising engineers discovered very soon that the theory was very insufficient in giving a satisfactory solution to all the manifold problems involved. It fails to account for the effect of the compactness of the sand on the intensity of the lateral pressure; it fails to account for the conspicuous absence of the hydrostatic distribution of pressure over the timbered sides of cuts; it fails to account for the influence of rainstorms on the lateral pressure of fine sand on retaining walls with well-drained backs; and for many other important phenomena familiar to the man who works in the field. Nevertheless, for more than a century, progress has been limited to the replacement of the numerical evaluation of Coulomb's formula by more expedient graphical methods. The causes of the frequent contradictions between theory and reality remained as obscure as they were at the outset, and the textbooks left it to the practising engineer to decide in which cases and to what degree a forecast of the theoretical pressure could be relied upon. That means that for more than a century the practical application of the theory retained all the characteristics of a gamble.

This unsatisfactory condition did not change until some 20 years ago, when the patience of the progressive members of the engineering profession seemed to be exhausted with the classical conceptions. The reasoning which initiated the reformation was very simple: it consisted of tracing the known inconsistencies back to their sources. The theory, as such, was found to be flawless, because it is exclusively based on the laws of mechanics

and there is no exception to the validity of these laws. Hence the source of existing contradictions between theory and reality can only reside in the basic assumptions (1)–(4) in the preceding list. The subsequent investigation of the basic assumptions furnished the following results.

By direct experiment it was found that the value ϕ in equation (1) is not identical with the angle of repose, except for dry sand in a loose state. In contrast to the angle of repose, which has always a value of about 34 degrees, the angle of internal friction ϕ can assume any value between the angle of repose and an angle greater than 40 degrees, depending on the degree of compactness of the sand. When I published this statement for the first time, some 15 years ago, I was charged by one of the participants in the discussion to be inadequately grounded in theory. The statement, it was claimed, must be wrong. Since that time, as a consequence of the rapid development of our methods of testing, the influence of compactness on the value of the angle of friction became a commonplace experience, which every experimenter nowadays takes for granted.

The influence of the presence of water on the stability and on the lateral pressure of sand was provided for by most engineers by the assumption that the water reduces the value of ϕ in equation (1) by 6 or more degrees. In contrast to this widespread opinion experiments established the fact beyond any doubt that the influence of water on the ϕ-value of sands is practically nil. The cause of the known effects of water on stability and pressure simply resides in the fact that the water carries one part, u_w, of the total normal pressure p which acts on the potential surface of sliding. Since the water has no shearing resistance, the presence of the water reduces the shearing resistance along an interface through a saturated sand from the value determined by the classical equation (1) to

$$s = (p - u_w) \tan \phi \quad . \quad . \quad . \quad . \quad (2)$$

Equation (1), as thus modified, eliminates the necessity for an assumption concerning the non-existing "lubricating effect" of water on sand. The value u_w was found to be equal to the unit weight γ_0 of the water, multiplied by the height h to which the water rises in a standpipe from the point to which equation (2) refers. Substituting in equation (2) the product $\gamma_0 h$ for the value u_w, we obtain

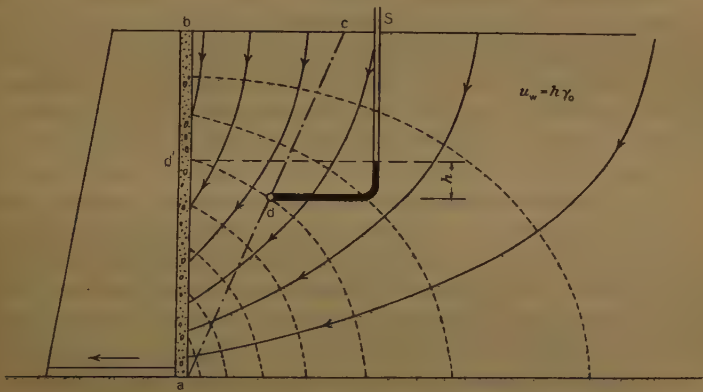
$$s = (p - \gamma_0 h) \tan \phi \quad . \quad . \quad . \quad . \quad (3)$$

Equation (3) also explains the influence of a rainstorm on the lateral pressure of fine sand on a retaining wall with a drained back. This effect is illustrated by *Fig. 2*. The drainage-layer which covers the entire back of the wall discharges the accumulating waters through weep-holes located above the base of the wall.

In humid climates fine-grained backfills are always in a very moist state throughout the year because the water is retained in the voids of the fill by capillary forces and the rate of evaporation is very low. If, however, a

standpipe is inserted (shown at S in *Fig. 2*) into such a soil during a dry period the water does not rise in the standpipe at all. This observation leads to a value of $\gamma_0 h = 0$. As a consequence, during the dry period equation (1) is valid although the voids of the soil are almost completely filled with water. As soon as a rainstorm starts, however, the state of partial saturation passes into one of complete saturation. The rainwater floods the surface of the fill and percolates through the fill towards the vertical drain. The paths followed by the water-particles can be computed by means of the formulas which are used for seepage computations in general. In *Fig. 2* these curves are represented by plain curves provided with arrows. The corresponding curves of equal standpipe-level are shown by dotted lines. According to the laws of hydraulics the water rises at every point d (*Fig. 2*) to the elevation of point d′, the intersection between the equipotential curve dd′ and the drainage-layer. Hence the effect

Fig. 2.



of the rainstorm is to increase the value u_w in equation (2) from zero corresponding to the state before the rainstorm, to $\gamma_0 h$ during and for some time after the storm. The total normal pressure p on the surface of sliding remains practically unaltered. As a consequence, according to equation (2) the storm produces a temporary decrease of the shearing resistance along the surface of sliding, which in turn causes a temporary increase of the lateral pressure. This temporary increase can amount to as much as 35 per cent. of the initial value of the lateral pressure. In order to avoid this undesirable by-product of rainstorms, we must modify the position of the drainage-layer such as to enforce, within the " wedge ", a vertically downward movement of the water. This results in a horizontal position of the lines of equal standpipe-level.

Thus our inquiry has led to a radical departure from the original conception of assumption No. 1. It removed some of the known contradictions between theory and reality. It demonstrated the non-existence of the

"lubricating influence" of water on sandy backfills, and helped to establish a conception which is compatible with both the laws of hydraulics and the physical properties of the materials involved. Thus it opened the way for understanding and controlling the influence of water on earth-pressure phenomena. This must be considered to be an accomplishment of outstanding importance, because in engineering practice difficulties with soils are almost exclusively due not to the soils themselves but to the water contained in their voids. On a planet without any water there would be no need for soil mechanics.

According to the second fundamental assumption of Coulomb's theory the surface of sliding should be plane. It was already known in Coulomb's day that the surface of sliding is curved, but knowledge concerning the importance of the error due to disregarding the curvature remained limited to a few special cases having no practical importance, and no progress was made until von Kármán, Jáky, Ohde, and others published the results of their successful efforts during the last 15 years. By means of advanced analytical methods, these investigators determined the real shape of the sliding surface for both the active and the passive case, on the assumption that the soil is on the verge of failure at every point in the wedge. The results are as follows: in both cases the surface of sliding consists of a curved lower part and a plane upper part, such as is shown in *Figs. 1* (*e*) for the passive pressure. For the active pressure the error due to the assumption of a plane surface of sliding never exceeds 3 per cent., and this is negligible. For the case of passive pressure on a lateral support with a rough back, however, the error can be greater than 30 per cent. The accurate methods which have been used by the investigators referred to are far too complicated for practical purposes. Yet, once the necessity was established for considering the curvature of the sliding surfaces in backfills subject to passive pressure, it was an easy matter to find an approximate method which serves the purpose. The approximate method involves replacing the curved part ad of the surface of sliding shown in *Figs. 1* (*e*) by a logarithmic spiral; the error associated with the use of this method does not exceed 3 per cent. Thus an important shortcoming involved in the classical theory has been eliminated.

According to the third assumption the shearing resistance of the supported earth should be fully mobilized along the entire surface of sliding. In a natural bed of earth, or in a fill deposited behind a rigid obstacle, the shearing stresses are insignificant compared to those corresponding to the state of failure. Hence the third assumption of Coulomb's theory can only be satisfied if the sliding wedge (abc in *Figs. 1* (*a*)) is given an opportunity for lateral expansion corresponding to the deformation which precedes the failure of a solid body. For retaining walls this expansion can only be achieved in conjunction with a lateral yield of the wall, and in cuts by the exposure of the working face during excavation combined with the compression of the struts and wedges. The amount of yield required for

mobilizing the shearing resistance of the soil is determined by the height of the supported face and by the elastic properties of the soil.

For fairly dense sands and sandy soils it was found that the required yield does not exceed a small fraction of 1 per cent. of the height of the face. As a consequence, in actual practice Coulomb's assumption can be satisfied without a noticeable yield of the lateral support. This empirical fact led superficial observers to the conclusion that the mobilization of the shearing resistance of the soil does not require any yield at all.

On the other hand, for soft clays the necessary movement is greater than that which can be tolerated for a retaining wall, and very much greater than the distance through which the struts in a cut can be compressed without buckling. Furthermore, from a study of the physical properties of clay it became evident that the shearing stresses in a supported bank are subject to changes in course of time, without change in the external conditions. The combination of these facts exclude the application of earth-pressure theories of any type to the computation of the pressure exerted by soft clay, and whatever hopes have been cherished in this respect during the past century must be buried. The pressure of clay on lateral supports can only be learned from experience, to be acquired by actually measuring the pressure in different cuts and correlating the results with complete quantitative data regarding the nature of the soils investigated. Very few measurements of that sort have ever been made. As a consequence our knowledge concerning this subject is quite inadequate, and the designer is free to choose between gambling and designing with an excessive factor of safety.

The fourth and last condition for the validity of Coulomb's theory involves the breakdown of the equilibrium of the soil in the entire space abc (*Figs. 1*) between the sliding surface and the lateral support. This in turn represents the condition for the triangular distribution of lateral pressure over the back of the support, such as shown in the right-hand diagram in *Figs. 1* (*a*). The corresponding centre of pressure is located at one-third of the height, *H*, of the support.

With an increasing knowledge of the mechanical properties of soils it was gradually realized that the type of failure specified by the fourth assumption of Coulomb's theory represents only one of many possibilities which exist in engineering practice. For mechanical reasons simultaneous breakdown in the entire wedge does not occur, unless the lateral support yields by tilting around its lower edge. In every other case—for example, that shown in *Figs. 1* (*b*), involving a yield by tilting around the upper edge of the support—the failure starts with a disintegration of the lower (shaded) part of the wedge and is associated with a slip along the upper part of the surface of sliding. As a consequence, the condition for the hydrostatic pressure-distribution ceases to exist. Instead of transmitting its weight on to the lower part of the wedge, the earth contained in the upper part "arches" between the surface of sliding and the back

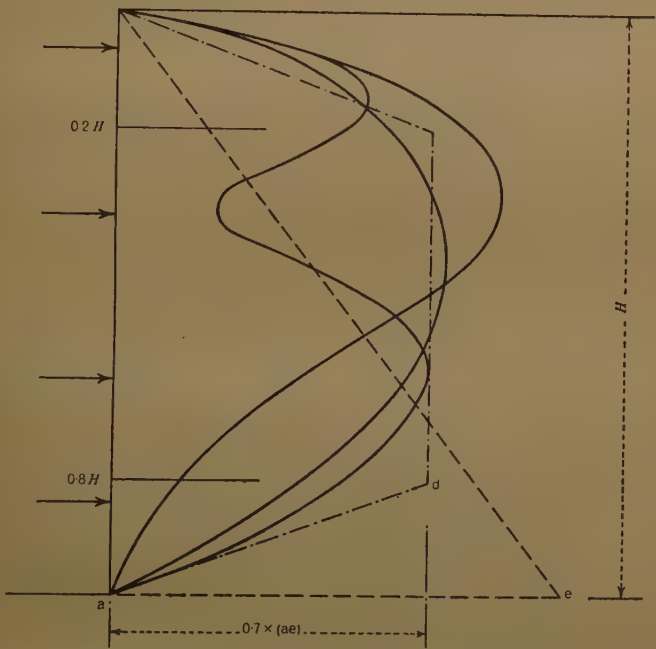
of the support, and as a result the lateral pressure on the upper part of the support increases at the expense of the pressure on the lower part. The arching effect also influences the shape of the surface of sliding. Instead of being almost plane, the surface exhibits very marked curvature, shown as ac_1 in *Figs. 1* (*b*). An understanding of this action leads to the conclusion that the pressure-distribution associated with the conditions shown in *Figs. 1* (*b*) should be approximately parabolic, the centre of pressure being located in the vicinity of one-half of the depth of the cut. In engineering practice the closest approximation to this condition should be expected in timbered cuts in sands.

The first time (some 4 years ago) that I explained this theory before an audience outside a classroom, I experienced anything but a benevolent reception. Fortunately, the discussions aroused the interest of representatives of the Siemens Bau Union, an organization which for many years had been engaged in the construction of subways in sand. Considering the practical importance of the issue the firm decided to measure the pressures in a cut 360 feet long and approximately 40 feet deep. Investigations were made at ten different sections, and the results are shown in *Fig. 3.* In spite of rather erratic deviations from the parabolic average, the results are remarkably consistent. In every one of the profiles the centre of pressure was located between 0·45 H and 0·55 H. Computing backwards from the measured total pressures, these pressures correspond to an angle of internal friction of the order of 40 degrees. This is equal to the known value of ϕ for similar sands in the densest state. Hence we are justified in concluding that the pressure-distribution shown in *Fig. 3* was associated with an almost complete mobilization of the shearing resistance of the supported sand. Quite recently Lazarus White made similar measurements in a deep cut in New York through very fine sand locally called "bulls' liver." The pressure-distribution which he observed does not differ in any vital respect from that shown in *Fig. 3.* Hence the theory which has led to the result shown in *Figs. 1* (*b*) has stood the crucial test of comparison with engineering reality.

If a lateral support yields by lateral deflexion between an upper and a lower belt of greater resistance, similar to the yield of the bulkhead represented in *Figs. 1* (*c*), the normal arching in the upper part of the wedge abc combines with a lateral arching comparable to that which occurs in a bed of sand above a yielding trap-door. The resulting distribution of lateral pressure must exhibit the characteristics shown in the right-hand diagram of *Figs. 1* (*c*). The bending moment produced by this type of lateral pressure is very much smaller than the moment produced by a pressure of equal total intensity having a hydrostatic distribution. The existence of this peculiar pressure-distribution was realized more than 10 years ago by Danish engineers from the fact that many ancient bulkheads along the waterfront of Copenhagen would fail if subjected to the bending moment which corresponds to a hydrostatic distribution of lateral pressure. At a

later date these findings were verified by the experimental investigation of Mr. J. P. R. N. Stroyer [1] in London. The practical conclusions derived from the discoveries of the Danish engineers are incorporated in the Danish Standard Specifications for the design of bulkheads. These specifications provide against a waste of material which is the result of assuming a non-existing hydrostatic distribution of pressure in bulkhead design.

Fig. 3.



The preceding summary of recent developments of knowledge pertaining to earth-pressure is far from complete, yet it may suffice to bring out the essence of what has been accomplished. The methods available 20 years ago for dealing with earth-pressure problems were based on assumptions which are seldom satisfied under field-conditions. The limits of applicability were almost unknown, and as a consequence any practical application bore the characteristics of a gamble. To-day these methods are superseded by a general procedure which covers a great variety of practical cases, with the exception of those involving the pressure exerted by typical clays. The rules for adapting this procedure to existing conditions are well established. The contradictions between theory and practice have ceased

[1] "Earth-Pressure on Flexible Walls." Journal Inst. C.E., vol. 1 (1935–36), p. 94. (November 1935.)

to exist. In the case of clays we know that under practical conditions the fundamental requirement of an almost complete mobilization of shearing resistance is not complied with. As a consequence, problems dealing with pressure exerted by clays under field-conditions are for ever beyond the scope of earth-pressure theories in general. In our future treatment of these problems we must choose between maintaining our present state of ignorance, and acquiring a stock of reliable experience by systematic measurements and identification-tests in the field.

All of these vital improvements have been realized as a result of the discovery that the intensity and the distribution of earth-pressure depends on many more factors than the authors of the older theories anticipated. Once the existence of these factors was recognized it was an easy matter to modify the theories accordingly. In a similar manner all other fields of earthwork and foundation engineering have been critically studied for the purpose of weighing the importance of ignored or neglected factors. The results of these investigations have been by no means less revolutionary than those in the realm of the classical earth-pressure theories.

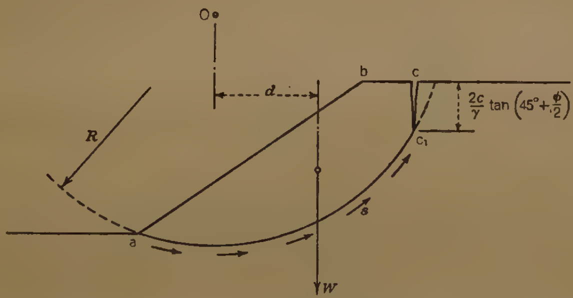
Stability of Slopes.

Closely related to the problems of earth-pressure are those relating to the stability of slopes. In older textbooks on civil engineering this subject is not treated, it being considered too simple. It was sufficient to prescribe that the slopes of fills and cuts should be established at an angle somewhat smaller than the angle of repose. The values of the angle of repose for different types of soils were assembled in Tables, regardless of the fact that every cohesive soil—most of the soils in nature are cohesive—can stand in vertical banks at a height depending on the nature of the soil.

Considering the arbitrary character of the data used as a basis for selecting a slope, it is not surprising that nature occasionally protested against this violation of its laws. Such was, for instance, the case some 20 years ago, in southern Sweden. Within a relatively short period several serious slides occurred along the railways of that part of the country, one of which caused the loss of 41 lives. In order to avoid any further accidents of this type the railroad administration appointed a committee, including amongst others Messrs. W. Fellenius and J. Olsson, to investigate the degree of safety of the existing slopes and to propose measures for eliminating the danger. The report of that committee was published in 1922. It contained the most complete analysis of landslide phenomena and of soils subject to slides which had been published to that time. It disclosed the fact that it is economically impossible to establish slopes in difficult country so as to comply with the usual requirements for safety of structures, and that we must be content with precautions against danger of slides wrecking trains. In their investigations the members of the committee also made

extensive use for the first time of what is known as the Swedish method for determining the degree of stability of a slope. The method is based on the empirical fact that the profile of the surface of sliding always approaches the shape of a circular arc. The principle of the method is illustrated by *Fig. 4*. To a depth $\frac{2c}{\gamma}\tan\left(45°+\frac{\phi}{2}\right)$, where c denotes the cohesion and γ the unit weight of the earth, the soil fails by tension; below that depth

Fig. 4.



sliding occurs along the curved surface ac_1, when the shearing resistance of the soil is exceeded.

Let W denote the total weight of the sliding earth,
„ $\bar{s}$ „ average shearing resistance per unit of area of the surface of sliding,
„ $l = ac_1$ „ distance ac_1 measured along the arc, and
„ G_s „ factor of safety against sliding.

Taking moments around the point O, we obtain

$$Wd = \frac{l\bar{s}R}{G_s},$$

or

$$G_s = \frac{l\bar{s}R}{Wd} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The position of the surface of sliding for which the factor of safety G_s is a minimum must be determined by trial and error.

If a slide occurs before or after the fill is brought up to its final height, equation (4) can be used for computing the value $\bar{s}$. The fact that the slide has occurred indicates that $G_s = 1$, and the other quantities contained in the equation can be easily computed. The value of $\bar{s}$ being known, the section of the fill can be modified so as to satisfy specified requirements for the factor of safety. This method has also been used, and always with success, for determining the counterweight required for stabilizing natural ground which has yielded under the weight of a fill.

In the case of earth structures or natural banks consisting of fine-grained soils having very little cohesion, the most dangerous conditions exist during rainstorms. In the case of embankments provided for the storage of water, the most critical conditions exist on the downstream side, when the reservoir is full. For a mass of earth having very little cohesion and subjected to seepage forces, the shearing resistance s can be computed by means of equation (3) : $s = (p - h\gamma_0) \tan \phi$. In this equation the total normal pressure, p, is determined from the weight of the earth and water located above the surface of sliding. In order to determine the hydrostatic pressure, $h\gamma_0$, in the water it is necessary to draw a flow-net similar to that shown in *Fig. 2*. The value of ϕ in equation (3) can be obtained from shear tests. Once the value of s is known for every point of the surface of sliding the average shearing resistance $\bar{s}$ can be determined by graphic integration. The balance of the computation is made by means of equation (4).

The determination of the shearing resistance of typical clays is far more difficult. The relation between the normal pressure and the shearing resistance is much more complicated than could have been anticipated 10 years ago, and even to-day our knowledge of this subject is still in a somewhat controversial state. The most important recent contribution was made in 1936 by Mr. M. J. Hvorslev[1].

If a submerged body of soil having little or no cohesion is in a loose state, an insignificant external or internal disturbance, such as might be produced by a mild earthquake or by a spontaneous subsidence within the earth, is likely to transfer the soil temporarily into a liquid state, causing it to flow on a horizontal surface. This phenomenon is known as a "flow-slide." In order that a material can flow upon suitable provocation, it must be loose enough to have a tendency to reduce to a denser state as a result of any type of deformation. If this requirement is fufilled, a spontaneous subsidence transfers part of the weight of the solid constituents to the water, thus considerably increasing the pressure u_W which acts in the water, while the total pressure in the mass of earth remains unaltered. According to equation (2),

$$s = (p - u_W) \tan \phi,$$

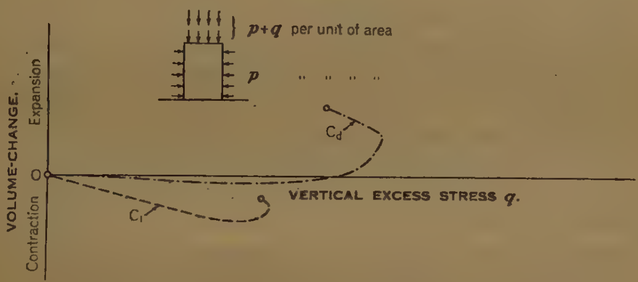
an increase of the pressure u_W in the water, at unaltered total pressure p, reduces the shearing resistance s, which has the same effect as if the internal friction of the mass were diminished. If this reduction is sufficient, the mass assumes the characteristics of a viscous liquid and retains this consistency until the excess water has drained away.

In a dense sand every deformation produces a tendency to expand. This excludes the possibility of a flow-slide. At some intermediate porosity the material neither expands nor contracts. Mr. A. Casagrande

[1] "Uber die Festigkeitseigenschaften gestörter bindiger Böden." Ingeniorvidenskabelige skrifter A. Nr. 45.

proposed the method illustrated by *Figs. 5* for determining this "critical porosity." A tri-axial compression test, in which it is possible to measure the volume-change produced by increasing the axial pressure q at unchanged hydrostatic pressure p, is employed. The observed data are plotted graphically. Tests are made on specimens of different porosities. The lower curve C_l in *Figs. 5* represents the test results obtained from a specimen having a high porosity, and C_d those from one having a low porosity. No flow-failure can occur unless the volume of the soil at failure is smaller than the initial volume. Hence in the case of a specimen possessing the critical porosity, the point on the curve corresponding to failure must be located on the horizontal axis. Judgment alone does not make possible a decision regarding whether or not the danger of a flow-failure exists. This was demonstrated a short time ago, at the expense of the taxpayers,

Figs. 5.



by the failure of a dam under construction in the U.S.A. Without warning, 10 million cubic yards of earth were lost within a few minutes. In order to avoid the recurrence of such accidents, the specifications for future structures of this type will provide that the porosity of the embankments is to be well below the critical value.

When a modern concrete-surfaced highway is constructed on a fill no appreciable settlements can be tolerated, as otherwise the cost of maintenance of the road surface becomes forbidding. The settlements of embankments of the same dimensions and made of the same materials decrease with increasing compactness due to their own weight. Mr. R. R. Proctor demonstrated by tests that the degree of compaction produced by a given method of compacting depends to a large extent on the moisture-content of the soil. As a rule, the compacting effect is greatest for a certain medium water-content, which can be determined by experiment. These observations led to fill-construction methods employing "moisture-content control." This method has become the universal practice in the United States in recent years.

In Germany methods for compacting sandy soils by vibration have been brought to a high degree of perfection under the leadership of Mr. A. Hertwig. It was found that vibration has by far the greatest compacting

effect if the frequency of vibration is equal to the specific frequency of the soil. This specific frequency ranges from 20 and 30 hertz. A few years ago a vibrator appeared on the market having a service weight of 24 tons. The vibration method is also successfully used for determining the relative degree of compaction of fills during and after construction.

In connexion with the stability of slopes it was mentioned that the danger of sliding is greatest when the earth located behind the slope is subjected to seepage forces. The mechanics of this influence are closely related to those of a dreaded phenomenon, the failure of dams due to piping.

Failure of Dams Due to Piping.

During my apprenticeship, a general contractor by whom I was employed built a concrete gravity dam on a gravel foundation. In those days there were no accepted rules for determining the depth of sheet-piles to be driven along the upstream and downstream edges of the body of a dam. As a consequence the depth of the sheet piles was determined by the chief engineer on the basis of his personal experience and his perusal of publications dealing with foundations of similar structures. Immediately after the reservoir was filled for the first time, the dam failed by piping. The middle part of the concrete section subsided to such a depth that the crest had to be located by means of a sounding rod. The sight of this catastrophe made a lasting impression upon me.

A few years later, in 1910, Mr. W. P. Bligh published his well-known rules for the design of dams on a porous foundation. Considering the state of our knowledge in those days, his contribution was of outstanding importance, and for many years it did not occur to me that there could be any flaw in the procedure. In 1919, however, after I had begun a systematic search for neglected variables, I also included the phenomenon of piping in the programme of my investigations. The results were surprising.

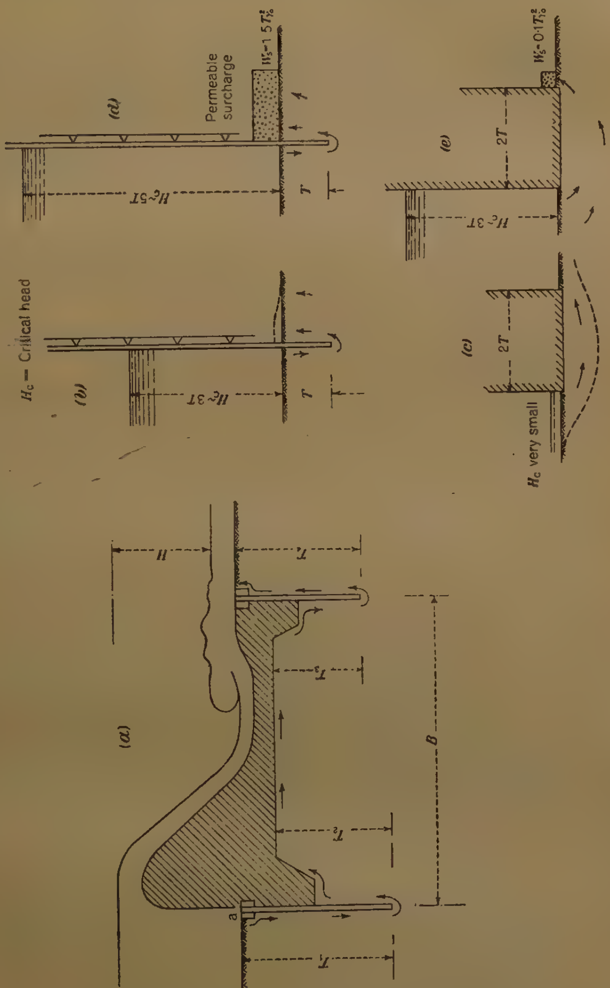
According to Bligh's theory the degree of safety of a dam against piping should depend only on the nature of the subsoil and on the average hydraulic gradient along the shortest line of creep, L_c, or the path traced by a water particle which enters the soil at point a in *Figs. 6* (*a*). He called the ratio

$$c = \frac{L_c}{H} = \frac{T_1 + T_2 + B + T_3 + T_4}{H} = \frac{B + \Sigma T}{H} \quad . \quad (5)$$

the percolation-factor, and specified that this value should be no less than 18 for fine sand and silt, and from 4 to 6 for a mixture of boulders, gravel, and sand. These empirical values were derived from a systematic analysis of all data that he could secure from a study of dams which had failed as a result of piping.

On superficial examination, Bligh's prescription seems to have all the

characteristics of a reliable empirical rule derived from broad experience. Upon closer investigation, however, one always finds some *a priori* assumption concealed within rules of this type. If such an assumption is faulty, the rule must also be invalid regardless of the amount of experience which



Figs. 6.

entered into its conception. Bligh's rule is based on the tacit assumption that the critical percolation-factor should depend upon nothing but the grain-size of the subsoil. The term "critical percolation-factor" defines the ratio between the length of the shortest line of creep, $L_c = B + \Sigma T$ in *Figs.* 6 (*a*), and the head H_c at which piping occurs.

In order to ascertain whether this assumption is justified, I built in

1919 a series of model-dams on identical sands with equal " shortest creep ", $L_c = 2T$. According to Bligh's assumption the critical head, H_c, should have been exactly the same for all these dams. Instead of confirming this assumption, the tests showed that the critical head, H_c, ranged between the extreme limits of $1{\cdot}5L_c$ for the type shown in *Figs. 6* (*b*), to almost zero for the type shown in *Figs. 6* (*c*).

Further investigations into the causes of this disconcerting result revealed the fact that the critical head, H_c, depends not on the average gradient along the line of shortest creep, but on the greatest hydraulic gradient, i_{max}, at which the water flows out of the ground. For a perfectly uniform subsoil and for a horizontal discharge-surface, the danger-zone is located at the downstream edge of the structure. At equal " shortest creep ", L_c, and equal head, the maximum hydraulic gradient can vary widely, depending on the cross section which has been selected for the foundation. As a result the critical hydraulic head, H_c, must also be very different. This is incompatible with Bligh's rule.

The " seepage force " j which acts on the sand at the danger-point was found to be equal to

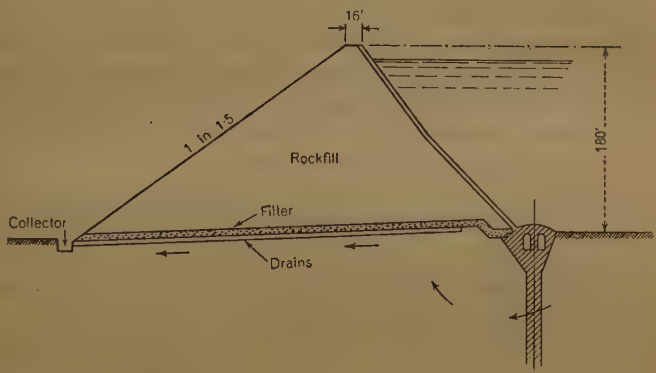
$$j = i_{max} \gamma_0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

per unit volume of the sand, the symbol γ_0 denoting the unit weight of water. Since this force was found to represent the mechanical cause of piping, it should be possible to increase the critical head H_c by opposing this force, which acts in an upward direction, by a weight acting downward. Since an impermeable surcharge diverts the danger-zone from its original location to the boundary of the surcharge, thus defeating the purpose of the surcharge, it must be made to consist of, or to rest on, an inverted filter. In order to check this conclusion, a supplementary series of tests was made. *Figs. 6* (*d*) and *6* (*e*) show some of the results. In the case shown in *Figs. 6* (*e*) an insignificant surcharge was found to suffice to increase the critical head H_c to many times the value obtained from tests performed on a model with a free discharge-surface. As a final check on the validity of the new conceptions an attempt was made to compute the results of various tests on model-dams in advance. The attempt was successful.

These discoveries opened the way to many practical applications. After having acquired some experience with inverted filters on minor structures, I designed a filter covering an area of 400,000 square feet for a rockfill dam in Algiers, a cross section of which is shown in *Fig. 7*. The body of the dam covers the outcrops of several strata of fine sand which, after the reservoir is filled, will furnish channels for an appreciable flow of seepage towards the base of the dam. Since the discharge-area will be covered with rockfill, incipient piping of the water in the sand cannot be detected until it is too late. The inverted filter serves to exclude the danger of underground erosion without inducing a dangerous concentration

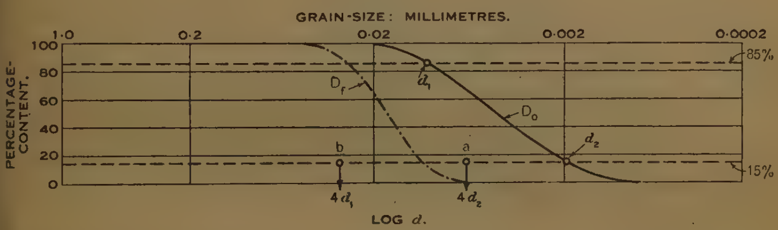
of flow-lines at some other point. In order to satisfy these requirements the filter-material had to be neither too coarse nor too fine. In *Fig. 8* are shown the results of tests which were made for the purpose of ascertaining the required grain-size. In this diagram, D_0 represents on a semi-logarithmic

Fig. 7.



scale the grain-size curve for the sand which is to be protected. As a result of the tests it was found that any sand will serve the purpose provided that its grain-size curve, D_f, intersects the 15-per-cent. line between the points a and b.

Fig. 8.



Assisted by the results of these and of similar investigations we are to-day in a position to eliminate the danger of piping at a moderate expense, simply by obstructing the escape-routes for the solid soil-constituents without interfering with the flow of seepage.

The Consolidation of Clays and Its Practical Consequences.

One of the many factors which has passed unnoticed in spite of its obvious practical significance is the time-element. From experience we know that the settlement of buildings or the pressure on the lining of tunnels may increase for many years, although external conditions remain un-

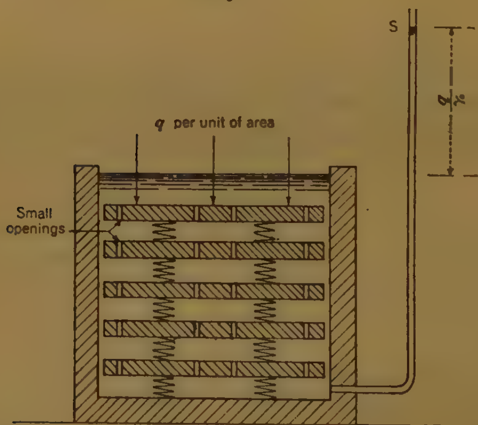
changed, yet the older textbooks on engineering may be searched in vain for equations pertaining to earths and foundations which contain the time-factor. The most conspicuous manifestation of the influence of time on settlement corresponding to a practically constant load is demonstrated by the settlement of buildings located above beds of medium or soft clays. The cause thereof was found to reside in the low permeability of clay soils.

For clays, as well as for any other porous materials, an increase of load produces a decrease of porosity as well as a decrease of thickness of layer. The relation between the pressure, q, and the void-ratio, e (ratio between the volume occupied by the voids and the solid), can be expressed with fairly close approximation by the equation

$$e = e_c - C_c \log \frac{q - u_w}{q_0}, \qquad (7)$$

where e_c denotes the initial void-ratio, u_w denotes the hydrostatic pressure in

Fig. 9.



the water-content of the clay, and C_c and q_0 are empirical constants. Since the voids of a clay in its natural state are almost or entirely filled with water, a decrease of the porosity of a clay is accompanied by an escape of a corresponding amount of water. Due to the low permeability of clay this escape can only occur very slowly. The mechanics of this "time-lag of consolidation" are illustrated by *Fig. 9.* This figure shows a cylinder with a system of perforated pistons maintained in their relative positions by two sets of spiral springs. The cylinder is considered to be completely filled with water. If a load q per unit area is applied to the uppermost piston, it cannot possibly descend to its ultimate position until the excess water has escaped from all of the various compartments. Hence at an early stage of the process of compression, the springs located in the lowest

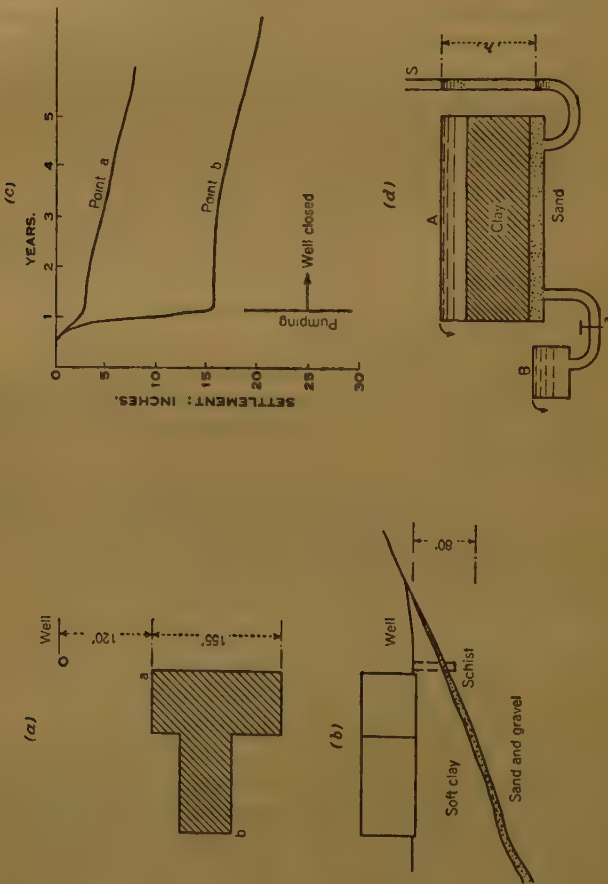
compartments will be under no compression. As a consequence, the entire load q must be carried by the water in these compartments. As a result the water rises in the standpipe S to an elevation $\frac{q}{\gamma_0}$ above the position which it occupied before the application of load. As time goes on the pressure in the springs increases, and a corresponding decrease in the pressure of the water takes place, as shown by a drop in the water-level in the standpipe. A similar process occurs in a layer of clay after loading its surface. The mechanics of this process can be expressed by a partial differential equation of the second degree, which is known as the fundamental equation of the process of consolidation. In this equation one of the variables is represented by the factor "time."

In order to demonstrate to unbelievers the existence of a hydrostatic excess pressure in the lower strata of a consolidating clay, some of my colleagues established an observation-well at the eastern approach to the San Francisco–Oakland Bay bridge, and named it "the monument to the theory of consolidation." The bottom of the well is located in soft clay at a depth of approximately 10 feet below the base of a fill. Due to the weight of the fill the underlying clay is in a state of progressive consolidation. As a consequence thereof the water rose in the casing several feet above the crest of the fill. This elevation in turn is several feet above the level of the bay. To the general public, including some engineers, this is an amazing phenomenon.

In 1919, while working on the theory of consolidation, I made the test shown in *Figs. 10* (*d*), p. 126. A layer of soft clay was deposited in a cylindrical vessel, above a layer of sand. The stopcock a was closed and the superincumbent water was maintained at level A. After the surface of the clay had ceased to subside, indicating complete consolidation of the clay under the influence of its own weight, stopcock a was opened, thus reducing the head of water in the layer of sand from h to approximately zero. The level of the superincumbent water continued to be maintained at A. By opening the stopcock the total normal pressure on a horizontal section through the clay was not altered, but the pressure u_w in the water-content of the clay was reduced from its original value to an appreciably lower value. According to equation (7), a decrease of u_w at an unaltered value of q should produce a decrease of the void-ratio of the clay, and thereby effect a new cycle of consolidation. The test results fully confirmed this theoretical forecast. As soon as the stopcock a was opened the surface of the clay commenced to settle again. It came to rest several weeks later, at an appreciably lower level, although the level of the superincumbent water was always maintained at A.

While this test was going on I was invited to co-operate in a case which proved to be an unintentional large-scale repetition of my laboratory test. The owners of a steam power-plant, *Figs. 10* (*a*) and (*b*), desired to open up a private water-supply for their power-house. For this purpose a well was

excavated through the clay down to a layer of water-bearing gravel, located between the base of the clay and the surface of the underlying rock. The owners were overjoyed with the abundant water-supply which was obtained, but their joy was tempered by their simultaneous discovery that the time-rate of settlement of the power-house had increased tremendously. Within a few months the corner adjacent to the well had



Figs. 10.

subsided more than 1 foot. In spite of the coincidence between the beginning of the pumping operations and the beginning of the increased rate of settlement, no one suspected a causative relation between the phenomena, and the incident was considered mysterious. Hence the owners were surprised when the settlement practically ceased, as illustrated by *Fig. 10* (*c*), after the well had been closed upon my suggestion. At about the time of this incident the Oslo "Underground" was under

construction. This project involved the excavation of a large tunnel through fissured rock located beneath a bed of soft glacial clay some 70 feet thick. Day by day water was pumped out of the tunnel and here again it was considered an incomprehensible act of providence that the beginning of the pumping operations coincided with the beginning of a progressive and very detrimental settlement of the buildings located above the tunnel. It was only at a later date, when construction was finished and the law-suits had started, that the theory of consolidation was remembered.

At the International Conference on Soil Mechanics and Foundation Engineering in 1936, Dutch engineers communicated observed data regarding the settlement produced by a lowering of the ground-water level for the purpose of excavating a pit 900 feet by 160 feet by 20 feet deep. The water was pumped out of a layer of sand located beneath a bed of soft clay, 20 feet in thickness. At a distance of 140 feet from the pit the subsidence of the surface amounted to 2 feet, and even at a distance of 2,800 feet the settlement was noticeable.

At the present state of knowledge the settlement due to pumping from sand strata located beneath beds of clay can be computed with a reasonable degree of accuracy from the results of soil-tests on undisturbed samples. There is no excuse for not anticipating such settlements, yet it seems to be extremely difficult for some engineers to perceive the mechanics of this relatively simple process. During the last 10 years the city of Mexico started to exploit, for water-supply purposes, some of the sand beds located beneath the area covered by the city, with disastrous consequences. Due to the resulting consolidation of the soft clay strata located above the sand, some parts of the city have already subsided more than 10 feet, causing serious damage to buildings and to the sewer-system. The causes of this subsidence are still the topic of lively controversies between the initiators and the adversaries of the theory of consolidation.

Settlement of Footings and of Raft Foundations.

Settlement due to the lowering of the ground-water level is one of the many vital topics passed over in silence in the older printed sources of information for civil engineers. The settlement of footings and of raft foundations is another. The instructions which the reader finds for the design of such foundations are limited to simple rules for selecting the "allowable bearing pressure", using as a guide either the general appearance of the soil (sand, stiff clay, etc.) or else the results of loading tests to be performed at the building site at the elevation of the base of the future foundation. The reader is free to believe that he need not worry about settlement if "allowable bearing pressures" are not exceeded. To say the least, he is most likely to believe that the settlement will be uniform,

provided that a uniform distribution of the pressure over all the loaded area is obtained.

Unfortunately neither one of these two widespread opinions is justified except under certain very limited conditions. According to the results of a general survey conducted by Mr. L. Casagrande in connexion with bridge-piers and abutments on German highways, the settlement of these structures ranged between the following limits:

Number of structures.	Description of soil.	Average soil-pressure: tons per square foot.		Settlement: inches.	
		From	To	From	To
21	Sand, gravel	1·5	3·0	0	0·4
31	Boulder clay, sand or gravel with clay	2·5	4·0	0	0·8
8	Clay, loess, loam, etc.	1·1	2·6	2	8
9	Silt	1·1	2·0	8	40

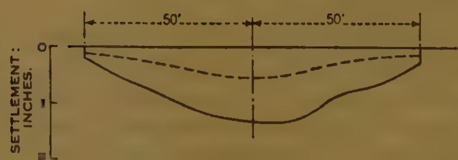
Since the conceptions regarding the admissible unit load for different types of soil are approximately the same all over the world, the above quoted figures can claim more than a local interest.

The assumption that a uniform load-distribution also involves a uniform settlement is not only in contradiction to the theory of Boussinesq,

Figs. 11.

(a)

(b)



published in 1885, but it is also in contradiction to practical experience. Some years ago I was called upon to design the foundation for the cylindrical tank shown in *Figs. 11* (*a*), which was to be built of welded steel plates. Since I expected, in accordance with the theory of Boussinesq, a bowl-shaped settlement, I considered it necessary to provide a very flexible

support for the bottom of the tank. As a consequence the bottom was placed on a feebly-reinforced concrete slab, 4 inches thick. *Figs. 11 (b)* show a section through the bottom at two different periods after the tank was filled for the first time. In spite of a perfectly uniform distribution of load the settlement exhibited the anticipated characteristics of a shallow bowl.

If we try to obtain a uniform settlement by placing the load on a rigid slab the soil-reactions must be very much higher at the rim than at the centre. Hence a raft will break unless it is designed to withstand the bending moments due to the difference in distribution of the load and of the reaction. This simple conclusion was verified at great expense by a party which tried to improve on the design of the foundation shown in *Figs. 11.* Since this party considered a 4-inch slab a rather inadequate support for a heavy structure, it was decided that the new tank should be placed on a heavily-reinforced concrete slab strengthened by ribs several feet in depth. The soil-conditions at the site of the new tank were similar to those shown in *Figs. 11*, except for the fact that the bed of soft clay was thicker and a greater settlement at the centre was therefore to be expected. Since the raft was too stiff to adapt itself to the bowl-shaped settlement it broke along two diameters, and both the tank and its precious liquid contents were lost.

"Allowable bearing pressures" always refer to the soil located immediately beneath the base of a footing or raft. To avoid too liberal an interpretation by economically-minded designers, however, the regulations usually specify that the satisfactory soil should extend to a depth of at least 10 feet below the base of a footing. If a satisfactory stratum rests on a bed of soft clay, and if, in addition, the area covered by the foundation is very large, the pressure exerted by the weight of the building through the sand on to clay will suffice to initiate an energetic consolidation of the clay. The results thereof are obvious. In one case which came to my attention, the footings of a building were established on a stratum of a very dense mixture of sand and gravel, "good" for at least 4 tons per square foot. The footings were designed for an "allowable bearing pressure" of 2·5 tons per square foot. At a depth of 23 feet below the base of the footings the gravel rested on a 50-foot bed of soft clay. Now, 40 years after the building was completed, the maximum settlement exceeds 3 feet and the differential settlement 2 feet. When the maximum settlement reached the 2-foot mark the owners began to worry, and requested that the foundation be strengthened. To accomplish this it was decided that the pressure at the base of the footings should be reduced from 2·5 to 1·5 tons per square foot. This was done, at great expense and with remarkable resourcefulness, by adding reinforced-concrete cantilevers to the existing footings. This engineering feat has been widely advertised; yet it induced no comment. No one seemed to be puzzled by the 2-foot settlement of an excellent gravel which took place under a load of 2·5 tons per square foot.

9

No one was perplexed by the fact that the 3-inch concrete pavement, located between the footings, remained practically intact in spite of a 2-foot subsidence of the adjoining footings. No one seemed to notice that the expensive reconstruction of the footings failed to make any impression on the trend of the time–settlement curves.

In order to obtain a clearer understanding of the principles involved in the rational design of foundations, we simply initiated the procedure which had been successfully used in connexion with earth-pressure phenomena. We again embarked on a search for " neglected vital factors " and achieved the following results. The design of foundations depending on " allowable bearing-pressure " methods are based on the tacit assumption that the settlement merely depends on the nature of the soil to a depth of 10 feet below the base of the foundation. In reality settlement depends on the character of the soil profile to a depth of at least 1½ times the width of a building, on the size and spacing of the footings, on the depth of a foundation, and—last but not least—on the location of the footings or of the loaded section of a raft with reference to the outer boundaries of the area covered by a structure. If the subsoil contains beds of clay the settlement is also a function of time.

Once the nature of the different factors which enter into a problem are known it is always possible to find some solution. Although our methods for accomplishing this purpose are still far from perfect, a number of examples of successful prediction of the magnitude, distribution, and time-rate of settlement of structures supported on clay foundations, are to be found in the Proceedings of the International Conference on Soil Mechanics and Foundation Engineering. Considering the type and number of misjudgements on record, the past state of our design methods for foundations was such that the mere realization of the nature of the factors involved in the problem of settlements represents an improvement of far-reaching practical significance.

Settlement of Pile Foundations.

If the top strata of the subsoil are too soft to carry the weight of a building the weight can be transferred to a lower stratum by means of piles. Textbooks prescribe either the use of pile-driving formulas or the making of loading tests for the purpose of estimating the bearing capacity of piles.

Pile-driving formulas are based on Newton's theory of impact. They represent the relation between the penetration produced by the blow of a hammer and the corresponding resistance Q_d counteracting the penetration of a pile. It is obvious that the quantity Q_d represents the dynamic resistance or the resistance of the pile to a very rapid penetration. In 1925 I published the results of investigations which showed that there

is no relation whatsoever between the dynamic and the static resistance of piles, except in the relatively rare case in which piles are entirely embedded in sand or in some other highly permeable material. Since that time these conclusions have been confirmed by the results of valuable pile-tests undertaken by the Whangpoo Conservancy Board in Shanghai under the supervision of Dr. Herbert Chatley, M. Inst. C.E., and also by many other investigations of a similar nature. Hence pile-driving formulas are dependable only in a few exceptional cases.

Loading tests furnish information concerning the relation between load and settlement for an individual pile. Both theoretical considerations and experience leave no doubt that there is no relation whatsoever between the settlement of an individual pile at a given load and that of a large group of piles having the same load per pile. The widespread opinion that a favourable outcome of a loading test excludes the possibility of serious settlements of an entire pile foundation ranks among the most incomprehensible and most detrimental prejudices in the field of civil engineering.

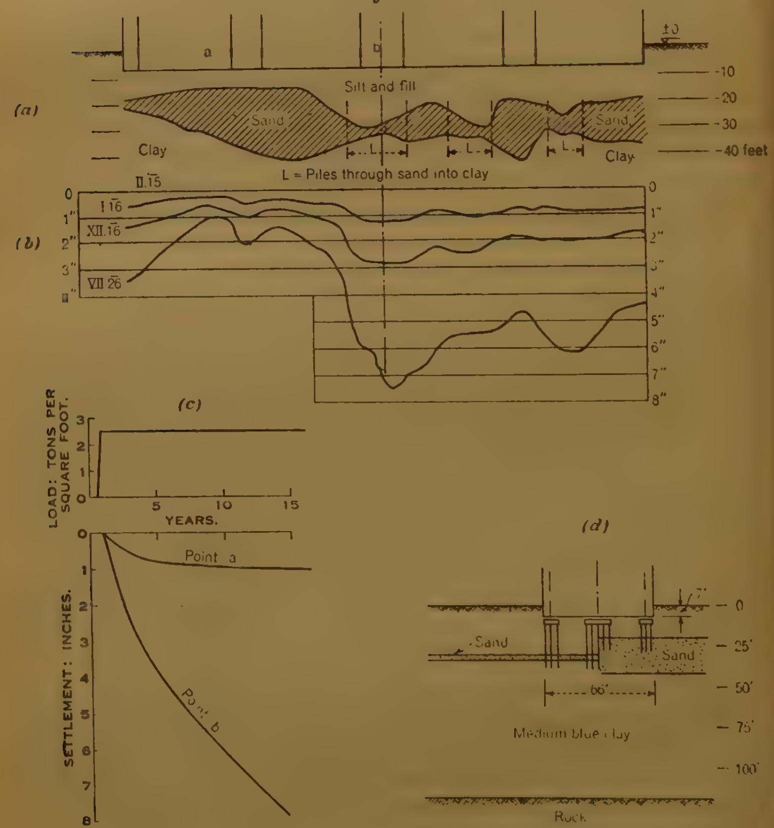
An impressive example of the practical consequences of this popular prejudice is illustrated by *Figs. 12* (p. 132). This monumental structure was intended to serve as an outstanding example of high-grade engineering. According to a statement published by the designers during the preliminary work, the foundation was designed so as to exclude the possibility of any settlement in excess of $\frac{1}{16}$ inch. In order to be sure of this prediction, the site was explored by means of twenty-one test borings and three test shafts. In addition, eighty test piles were driven and thirty-eight loading tests were performed. On the basis of the results of the loading tests and the pile-driving records the designers modified the pile-driving formula so as to adapt it to the subsoil conditions in existence at the site of the building. Finally the piles were laid out in such a manner that every pile carried exactly the load required to produce a settlement of $\frac{1}{16}$ inch. The results of these costly and painstaking efforts are shown in *Figs. 12* (*b*) and (*c*): 15 years after construction was completed the maximum settlement was more than 8 inches, and the differential settlement exceeded 7 inches. Even to-day, whenever I visit the building, I notice cracks which I have not observed before.

In 1928 I received an emergency call to visit a building under construction which was said to have started subsiding at an alarming rate, for perfectly incomprehensible reasons. The foundation rested on several thousand wooden piles from 50 to 80 feet long, driven to refusal in a bed of sand. Each pile carried one-third of the load required to produce a [illegible]-inch penetration obtained from a loading test. A near-by bridge pier having a similar foundation had not moved during the 50 years of its existence, yet the building went down and the settlement rapidly approached the 1-foot mark. The owners contemplated abandoning the site and the consulting engineer responsible for the design was confined to a hospital

with a nervous breakdown. The cause of the settlement resided in the gradual consolidation of a 30-foot layer of glacial clay, located at a depth of 100 feet below the surface and from 20 to 40 feet below the points of the piles. The curves of equal settlement were practically identical with those computed on the basis of the theory of Boussinesq.

Since that time there has been hardly a single year that I have not

Figs. 12.

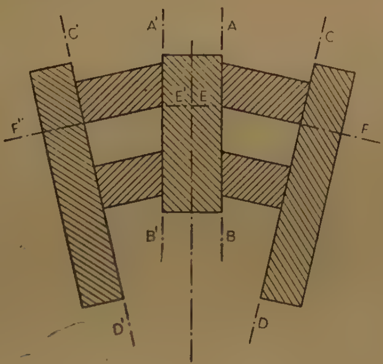


experienced similar tragedies. One of the most instructive cases came to my attention a few months ago. It is illustrated by *Figs. 13* to *15* (pp. 133 *et seq.*). The building, a beautiful monumental steel-framed structure, rests on 26-foot piles driven to refusal in a thin bed of sand, which transfers the load from the piles on to the surface of a bed of clay 160 feet thick. The bearing capacity of the individual piles is more than adequate and the settlement of the building is exclusively due to the consolidation of the clay. The

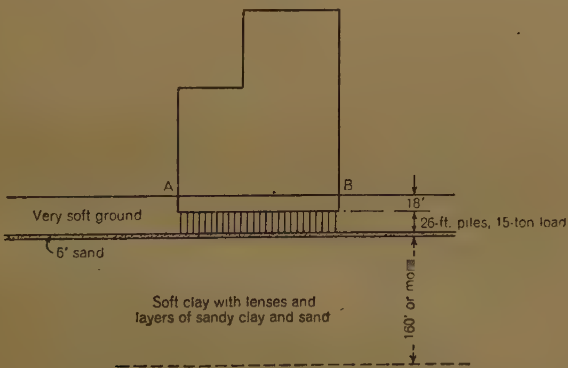
settlement is still increasing. At the left in *Fig. 14* are shown the curves of equal measured settlement, and at the right are shown the curves of equal settlement computed on the assumption that the building is perfectly flexible. In a general way the weight of the building produced the bowl- and trough-like depressions characteristic of settlements due to loading the surface of cohesive soils. The difference between the real and the

Figs. 13.

(a)



(b)

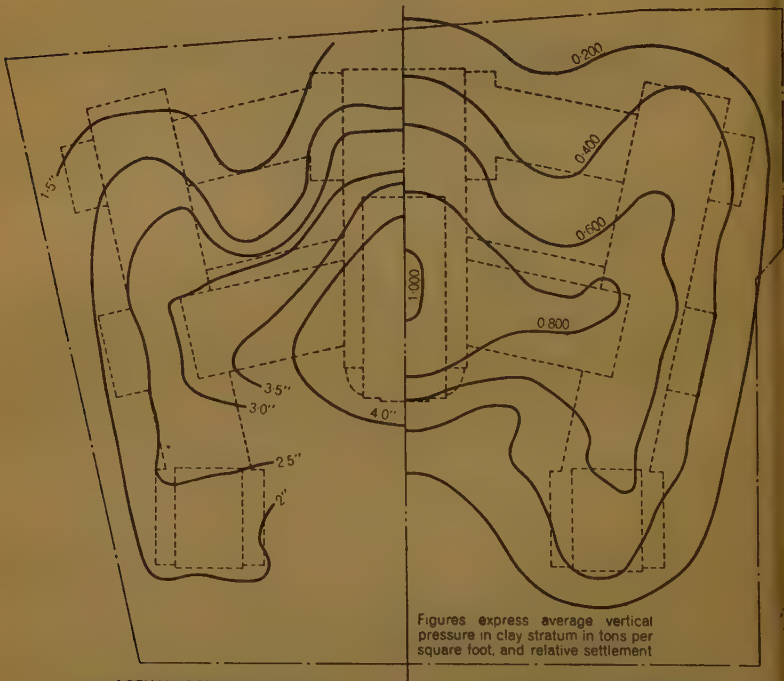


computed curves is essentially due to the influence of the stiffness of the building on the subsidence. This influence also shows up in the profiles in *Figs. 15.* The stiffness effects a reduction of the curvature of the warped surface, but the average settlement is unchanged. According to my comments on the settlement of the tank shown in *Figs. 11* the differential settlement cannot possibly be reduced without transferring part of the load from the centre towards the periphery of the loaded area. This

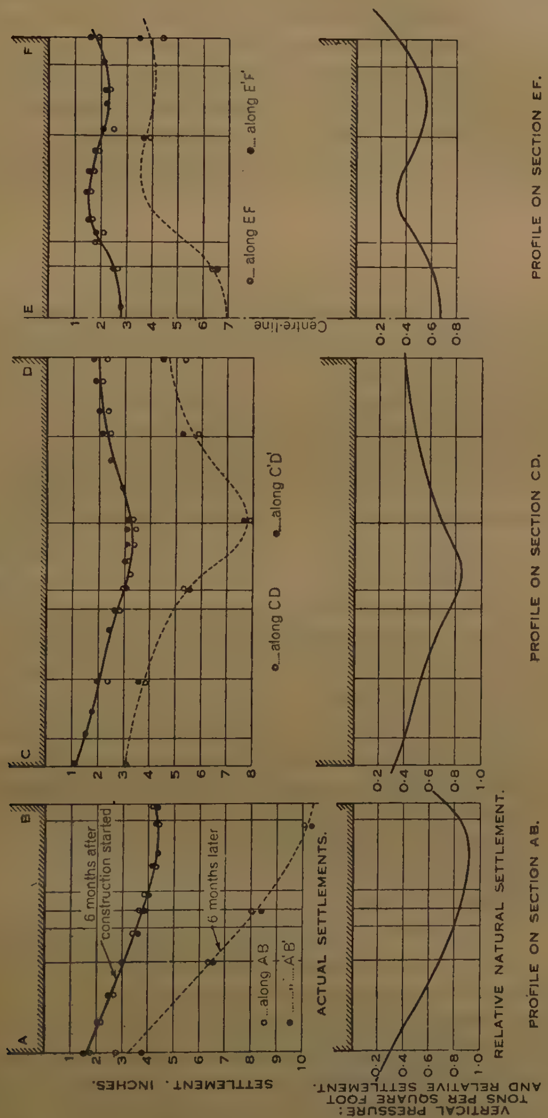
transfer is accomplished by the structure itself. If the structure is not sufficiently strong to withstand the stresses associated with the transfer a failure occurs, as did occur in the successor to the tank shown in *Figs. 11* This reasoning leads to an important conclusion concerning steel-framed buildings in general.

According to the accepted rules for the design of steel-framed buildings the members of the structure are designed on the assumption that the building rests on a perfectly rigid base. If the subsoil is compressible the

Fig. 14.



base of the building tends to assume the shape of a shallow bowl, yet the stiffness of the walls resists this tendency to a certain extent. This brings about a transfer of part of the weight of the building from the inner columns to the outer ones. As a consequence the stresses in at least the lower part of the steel frame are apt to be very different to those that the designer anticipated. This condition exists in every steel-framed building which has been constructed above beds of clay. Such a condition is a potential source of danger and is incompatible with the fundamental rule that the factor of safety for every member of a structure should be approxi-


A
B
C
D
E
F
6 months after construction started
6 months later
o ...along AB
• ..." ...A'B'
o...along CD
•...along C'D'
o... along EF
•... along E'F'
Centre-line
SETTLEMENT. INCHES.
ACTUAL SETTLEMENTS.
VERTICAL PRESSURE: TONS PER SQUARE FOOT AND RELATIVE SETTLEMENT.
RELATIVE NATURAL SETTLEMENT.
PROFILE ON SECTION AB.
PROFILE ON SECTION CD.
PROFILE ON SECTION EF.

Figs. 15.

mately the same. Thus far, nothing is known concerning the percentage of deviation of the real stresses from the theoretical stresses. It would therefore seem worth while to secure some pertinent information.

If the order of magnitude and the distribution of the future settlements are known in advance, it is always possible, by some means, to avoid harmful effects, and—last but not least—to eliminate the danger of the designer being criticized for carelessness and incompetence. For this reason it is advisable in the design of foundations to take advantage of whatever possibilities exist for estimating the settlement in advance.

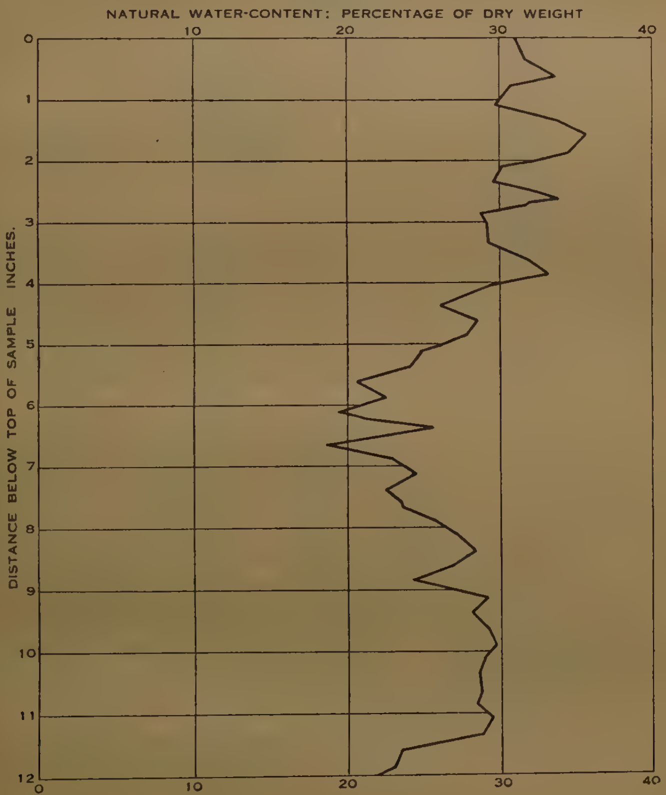
Sampling and Testing.

The fundamental requirement for an adequate forecast of settlement is an intimate knowledge of the compressibility of all clay strata contained in the subsoil down to a depth equal to at least 1½ times the width of the area covered by a building. More than 20 years ago, when engaged in the survey of landslides along their railroads, the members of the Swedish Geotechnic Commission noticed and described the softening influence of "kneading" a clay at unaltered water-content. At a later date Mr. A. Casagrande discovered a similar influence of "kneading" on the resistance of clays to compression while laterally confined. As a result of these discoveries it became necessary to develop methods for securing undisturbed samples. When I made my first subsoil explorations in the United States in 1925, I shocked a boring contractor by my request for a few "dry samples" of clay, to be obtained by driving a 1½-inch sample-tube with a bevelled edge into the bottom of a drill-hole, and by my request that the tube with its contents be sealed by means of paraffin immediately after recovery. When I returned to the United States in 1936 my original and very unsatisfactory method of sampling had been superseded by elaborate and ingenious procedures for furnishing almost undisturbed samples up to a diameter of 5 inches. The primitive methods employed in 1925 had practically disappeared. A similar development took place in Germany, France, Denmark, and many other countries. To-day in all of these countries a 5-inch undisturbed sample can be ordered without any further comment, and can be obtained at a reasonable price.

Another important development took place in the methods for selecting representative samples for testing purposes. In 1925, under the illusion that soil strata really are fairly homogeneous, I had the habit of requesting one "dry sample" for every 5 or 10 feet of test-borings through homogeneous strata. Since that time the painstaking investigations of Mr. A. Casagrande have destroyed my cherished illusion. No longer is there any doubt that homogeneous beds of clay are very rare. *Fig. 16* shows the variations of the natural water-content within a single foot of what I considered in 1925 to be a homogeneous clay. Due to the universal

absence of homogeneity, the essential prerequisite for selecting representative samples consists in securing complete data on the variation of at least one property of the soil along several vertical lines. The samples for the more elaborate soil tests are then selected in such a manner that the properties of the most frequent soil types are determined. The weighted average of the test results is obtained by statistical methods.

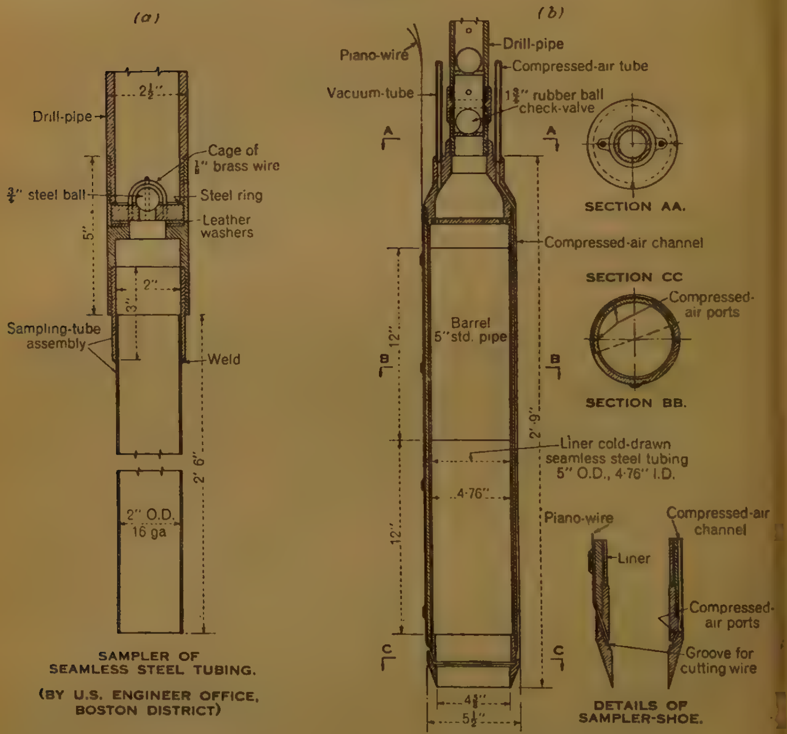
Fig. 16.



The " variation survey " described previously is accomplished by means of exploratory drill-holes. These furnish continuous samples approximately 2 inches in diameter. The tool for securing these samples is shown in *Figs. 17* (*a*) (p. 138). Immediately after recovering a sample the ends of the tube are sealed with metal disks and paraffin, and the samples are then stored in a humid room. Prior to testing the tubes are cut into 6-inch sections and for each section the index-property, for example the natural

water-content, or the natural water-content and the corresponding compressive strength, are determined. *Figs. 17* (*b*) show one of the many tools which are used for securing 5-inch undisturbed samples for the final soil tests. It is a double-barrelled sampling device. The inner barrel or liner is removable and is used as a container for preserving the sample during shipment and storage. At present Mr. J. Hvorslev is preparing a

Figs. 17.



SAMPLER OF SEAMLESS STEEL TUBING.
(BY U.S. ENGINEER OFFICE, BOSTON DISTRICT)

SOIL SAMPLER BY CASAGRANDE, MOHR, AND RUTLEDGE.

detailed report on the problems and current methods of sampling soils and the equipment employed. This report is being prepared for the Committee on Sampling and Testing, Soil Mechanics and Foundations Division of the American Society of Civil Engineers.

A short time ago I was offered an opportunity to organize a large-scale soil-exploration project for the Chicago Subway Authorities. This project may serve as an example for general procedure. The proposed length of the

subway is 40,000 feet. The construction requires the use of compressed air for tunnelling through soft clay at a depth of approximately 50 feet below street-level, and for several river-crossings. The "variation survey" includes about one hundred and fifty exploratory borings to a depth of 10 feet below the bottom of the invert. The borings are being made by a contractor on a cost-plus basis, and the price ranges between $1·50 and $2·50 per linear foot. The selected index-properties of the clay include the resistance to the penetration of the sampling tubes, the natural water-content, and the compressive strength. The tests are being made in a laboratory which was established by the subway authorities solely for that purpose. The personnel consists of one engineer, well trained in soil mechanics, who is in charge of the soil survey, and ten assistants. The soil survey is being supplemented by surveying daily the levels on a network of reference-points located above the different sites of underground excavation. The results of the soil investigations serve, first of all, as a basis for the design of the cross section of the future structure; they also serve to provide the contractors with detailed information on the soil-conditions, prior to submitting their bids, and to prevent unwarranted generalization on the basis of experiences gained during previous construction. Experience acquired in one section of the subway concerning the effect of the tunnelling operations on the street-surface and on adjoining buildings can only be used as a reliable guide in the construction of another section if the soil-conditions at the two sites are fairly similar. On superficial inspection the Chicago clay appears fairly uniform, yet the investigations have shown that the properties of the clay vary surprisingly between wide limits, both in a horizontal and in a vertical direction. The results of the survey afford no protection against blow-outs through old drill-holes or through locally-developed sand or gravel seams. They do, however, exclude the risk of cutting unawares into exceptionally soft beds of clay of considerable thickness. They permit a contractor to take full advantage of whatever experience has been gained during previous construction, and they protect the subway authorities against future charges of careless procedure. Compared to the capitalized value of these advantages the costs of this soil survey are negligible.

Legal Aspects of the Recent Developments.

While discussing the different branches of applied soil science I quoted numerous incidents involving damage to property. In all these cases damage was due to the fact that soil had behaved in a manner different to that which the designers had anticipated. Such cases invariably result in the embarrassing question: who is responsible?

At the beginning of the century the engineer was entitled to consider

the misbehaviour of his structure as a deplorable "act of God", and he was able to justify his claim by producing textbooks containing the essence of the knowledge of contemporary authorities. Since these authorities hardly mentioned the existence of settlements or the possibility of piping, no one could blame the average engineer for ignoring or misinterpreting these phenomena. The engineer made load tests and he conscientiously followed the accepted practice of keeping the soil-pressure below the tabulated and time-honoured "allowable bearing pressures." If a detrimental settlement occurred in spite of all these precautions, he was entitled to consider his case an abnormal one and to decline the responsibility.

However, as soon as some members of the profession acquire the capacity for predicting a phenomenon with such a degree of precision as is shown in the settlement diagram in *Fig. 14* (p. 134), and in many other published records as well, the phenomenon ceases to deserve the title of an "act of God." The same holds true for the effect of pumping on structures adjoining a well, for the failure of underpinning operations to accomplish their purpose, for the failure of earth dams, and for many other phases of practical earthwork engineering. To-day there is still some justification for the excuse that the methods for dealing with these problems are new and therefore only familiar to a small group of specialists. One cannot blame a physician in New Orleans for having failed to save the life of a patient by means of a method which had recently been developed in the Rockefeller Institute in New York. This excuse, however, will certainly not hold for ever, and the time is approaching when the Courts will decide against the designer who refuses to take notice of the existence of soil mechanics. Hence it is in the vital interest of all members of the profession to become familiar with the subject by active participation in the efforts to broaden our knowledge in the backward domain of civil engineering.

Conclusion.

One of the most important results of these efforts consists in establishing a theoretical insight into the mechanics of earth-pressure, settlement, and piping. This goes far beyond the boundaries of the primitive conceptions embodied in the textbooks published at the beginning of the century. Considering these scientific achievements we are entitled to ask: to what extent can this theoretical insight replace practical experience?

Instead of answering this question myself, I quote Dr. William Anderson: "Since 1848 the supremacy of theory over rule of thumb has gradually but surely asserted itself, though, at times, the want of common sense and experience in the application of abstract principles . . . has led to disasters quite as serious as those which arose from want of theoretical

knowledge; and in this respect the competent and successful engineer will still show himself as the man who in his work is careful to make theory and practice walk side by side, the one ever aiding and guiding the other, neither asserting undue supremacy."

There is no doubt in my mind that the mastering of our youngest science will greatly increase the capacities of the experienced engineer. My opinion has repeatedly been confirmed by the statements of seasoned practitioners who were open-minded enough to appreciate and to digest the fruits of our labours. At the same time I cannot help feeling deeply concerned about the self-confidence inspired in many members of the incoming generation of engineers by our knowledge, because (again quoting Dr. Anderson) "There is a tendency among the young and inexperienced to put blind faith in formulas, forgetting that most of them are based upon premises which are not accurately reproduced in practice, and which, in any case, are frequently unable to take into account collateral disturbances, which only observation and experience can foresee, and common sense provide against."

In 1893 this remark was aimed at the incoming tide of theoretically-trained structural engineers. To-day, in 1939, these golden words should be framed, to adorn the wall of every room in which research in soil mechanics is carried on. To accomplish its mission in engineering, science must be assigned the role of a partner and not that of a master.

Mr. Raymond Carpmael, in proposing a vote of thanks to the Lecturer, said that, quite apart from the immense value of what they had heard, the members were bound to have been impressed by Dr. von Terzaghi's personality and by his very clear exposition of the subject. They had also been brought to realize what an immense amount of work Dr. von Terzaghi had done, and was doing, to advance the study of soil mechanics. He felt that The Institution as a whole, and in fact all engineers, would profit very much by a study of the Lecture.

Dr. Herbert Chatley, in seconding the vote of thanks, explained that he had first come into contact with Dr. von Terzaghi's work some 13 years ago when struggling with the problems of clay at Shanghai. He had been presented with a copy of Dr. von Terzaghi's book, and immediately many of those problems had become capable of solution. Dr. von Terzaghi was very modest in describing the part he had played in the development of the science of soil mechanics, because in fact there was no important advance in which he had not played a predominant part in guiding thoughts into new channels. His study was a study of the motion of water on materials. It was because that subject had not been sufficiently considered before that his work had proved so illuminating. It had become a new science, to which his name, and almost his name alone, was attached.

The vote of thanks was carried with acclamation.

Dr. von Terzaghi, after expressing his appreciation of the vote of thanks, said that he hoped that his Lecture would give an additional impetus to the co-operation between the students of soil mechanics in Great Britain and abroad.

ORDINARY MEETING.

9 May, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

HAROLD MONTAGUE ALEXANDER, Jun., B.Sc. (Eng.) (*Lond.*).
GEORGE ANDREW-MARSHALL.
HARRY LAWRENCE BUNTING, M.C., B.Sc. (*Birmingham*).
CHARLES BUSFIELD.
GEORGE CARLYLE, M.C., B.A., B.E. (*Royal*).
THE HON. MANOCKJI NADIRSHAW DALAL.
ERIC LAUNCELOT EVERATT.
PHILIP HAVILAND HAVILAND, B.Sc. (*Witwatersrand*).
WILLIAM EDWARD HOGG.
JOSEPH WILLIAM HUSBAND, B.Eng. (*Sheffield*).
MALCOLM REGINALD JAMES.
WILLIAM ARNOLD JOHNSON, M.A. (*Cantab.*).
JOHN RUTHERFOORD D'OLIER LEES.
ARTHUR ALBERT ROWSE, B.Sc. (Eng.) (*Lond.*).
WILLIAM FREDERICK WEGENER.

And had admitted as

Students.

ALLAN CROFTON ROLLESTON ALBERY, B.A. (*Cantab.*).
ARTHUR ANDREWS, Jun., B.Sc. (*Glas.*).
TOM BASSETT.
PETER BRIGHT.
CHARLES BUCK.
WALTER JOHN BYGOTT.
ALAN CHAPMAN.
ROBERT FINDLAY COUPERTHWAITE.
JOHN VILLIS DIBBLE.
GEORGE MERVYN DICKSON.
VINCENT HUGH FENECK.
JOHN FRANCIS FLEMING.
YACOUB KIROLLOS GAYED.
FERGUS GILLIGAN.
JOHN ARTHUR GLEN.
ROBERT GEORGE GOURLAY.
LESLIE WARD HAINES, B.Sc. (*Birmingham*).
FRANK BARKER HAYES.
RICHARD GEORGE HEAD.
ERIC SHELDON HIBBERT.
PETER RICHARD HILL.
REGINALD HOWELL.
FRANCIS WILLIAM HULSE.
PHILIP SCOTT HUTCHINSON.
FRANK JAMES JOHNSON.
LINDSAY ANGUS ANDERSON LANG.
JOHN PATRICK NORTH LEWIS.
WILLIAM JOSEPH HERBERT LYNCH.
NEIL MACALISTER.
NEIL JOHN MILLER.
PETER BARRY MILNE.
HUGH O'DONNELL, B.Sc. (*Edin.*).
KANDASAMY RAMANATHAN.
BERNARD ESMOND SELBY RANGER.
GRAHAM RAVENSCROFT.
COLIN HARBIN ROBINSON.
JAMES ROSS.
WILLIAM CROSBIE MCGLASHAN RUSSELL.
DUDLEY ALAN SHAW.
JACK ARTHUR SHIPSTON.
ROBERT ARTHUR SMITH.
RODERICK ALAN STEPHENSON, B.Eng. (*Liverpool*).
ARCHIBALD STEVENSON THOM, B.Sc. (*Glas.*).
GWYNDAF EMUS THOMAS.
ROBERT FRANKLAND WALSHAW.

The Scrutineers reported that the following had been duly elected as

Members.

THE HON. HUGH TERENCE DE BURGH BINGHAM.
WILLIAM ALEXANDER ROGERSON.
ERNEST MORGAN STIRLING, B.A. (*Cantab.*).

Associate Members.

DAVID JAMES BLAIKLEY, B.A. (*Cantab.*), Stud. Inst. C.E.
HUBERT FRANCIS BRYANT, B.A. (*Cantab.*), Stud. Inst. C.E.
ERIC HARRY BURTON, Stud. Inst. C.E.
DONALD PRENTICE CARTWRIGHT, Stud. Inst. C.E.
REGINALD HERBERT CHAPLIN, Stud. Inst. C.E.
WILLIAM LANDMAN CHAPLIN, Stud. Inst. C.E.
THOMAS LATHAM COFFIN, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
DONALD FRANK COOCH, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
SAMUEL GEARY COTTON, B.A. (*Cantab.*), Stud. Inst. C.E.
ALBERT DE SOUZA, B.A., B.E. (*Bombay*).
ALLAN LEONARD FLAY, Stud. Inst. C.E.
COLIN ALEXANDER GORDON, Stud. Inst. C.E.
WESLEY HANCOCK, M.Sc. (*Birmingham*), Ph.D.
JACK MALCOLM HARNESS, M.Sc. (*Manchester*), Stud. Inst. C.E.
ADAM HOPE, B.Sc.Tech. (*Manchester*), Stud. Inst. C.E.
FRANK WILLIAM INGLE.
MOHAMED MOHSAN KHAN, B.Sc. (Eng.) (*Lond.*).
ALEXANDER GEORGE ANGUS MACKIE, B.Sc. (*Aberdeen*), Stud. Inst. C.E.
WILLIAM MICHAEL MATHERS, B.Sc. (*Leeds*), Stud. Inst. C.E.
ARTHUR MORANT, B.Sc. (Eng.) (*Lond.*).
HUBERT GERARD O'CONNOR, M.E. (*National*).
AUBREY ARTHUR PHILLIPS, Stud. Inst. C.E.
JOHN HUBERT REYNOLDS.
ARTHUR FIELDING RICHMOND, Stud. Inst. C.E.
ALAN SPENCER ROBINSON, B.E. (*Sydney*), Stud. Inst. C.E.
ROBERT ROSCOE, Stud. Inst. C.E.
CHARLES ANTHONY SERPELL, B.Eng. (*Sheffield*), Stud. Inst. C.E.
CHARLES BERNARD STONE, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
BARDICK SUKIAS, B.Sc. (Eng.) (*Lond.*).
ERIC TRANTER, M.Sc. (*Bristol*), Stud. Inst. C.E.
JAMES HAY WALKER, Stud. Inst. C.E.
WILLIAM WARDROP, B.Sc. (*Glas.*), Stud. Inst. C.E.
DAVID BOWE WATERS, B.Sc. (*Durham*), Stud. Inst. C.E.
WALTER REGINALD NEEDHAM GRUFYDD WILLIAMS, Stud. Inst. C.E.

ANNUAL GENERAL MEETING.

9 May, 1939.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Notice convening the Meeting was taken as read, as well as the Minutes of the Annual Meeting of the 10 May, 1938, which were confirmed and signed by the Chairman.

The following Report of the Council (pp. 149 *et seq.*) upon the Proceedings of The Institution during the Session 1938–39 was read, the Statement of Accounts (pp. 168–178) being taken as read.

The President moved—That the Report of the Council be received and approved, and that it be printed in the Journal of The Institution.

Sir Clement Hindley, Vice-President, seconded the motion.

Mr. J. S. Wilson, in discussing the motion, remarked that many members were very interested in the Proceedings and other publications of other engineering Institutions and societies; would it be possible to have the current numbers of the publications of such societies available for reference in the Main Library? At present he believed that such publications were kept in the Upper Library.

The President promised to look into the matter, but said that he thought that the current numbers of such publications were kept in the Main Library for a time and were removed for binding in due course.

Mr. H. G. Lloyd said that last year he had ventured to make a few minor suggestions with regard to the speeches of those taking part in Discussions, with a view to facilitating the ease with which they could be heard. He was pleased to see that those were approved and had been issued during the past session. He would like to mention, however, that, whilst there was a tendency for members to speak more than was formerly the case, there were still some who read their remarks. The result was that occasionally remarks were read that had already been made by a previous speaker, because obviously it was difficult to cut out those portions of a written speech which had already been said by someone else. He would therefore venture to suggest that some steps should be taken at the commencement of each session to remind members of the regulations which had been printed in the Journal.

There was one further point. It might have been noticed by members that evening that there was some resonance from some part of the Lecture-theatre. It was especially noticeable when the Secretary was reading the early part of the Report, but after a time it disappeared. That sometimes

happened with various speakers, and he was wondering whether some steps could be taken to reduce it.

The President, commenting on what had been said about speakers who read their remarks, said that it was true that it was very frequently done, and that there would be a general feeling that it was to be discouraged. He thought that the Council would give consideration to the steps which could be taken to make the regulation to which Mr. Lloyd had referred more effective.

With regard to the resonance from the Lecture-theatre, it was not due to the theatre itself; he thought that, since all the sloping seats had been taken out that evening, the acoustic conditions might have been altered.

The Meeting then resolved—That the Report of the Council be received and approved, and that it be printed in the Journal of The Institution.

The Scrutineers reported the election of the Council for 1939–40 as follows [1] :—

President.

Sir CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A.

Vice-Presidents.

Maurice FitzGerald Wilson.
Sir Leopold Halliday Savile, K.C.B.
Professor Charles Edward Inglis, O.B.E., M.A., LL.D., F.R.S.
Sir John Edward Thornycroft, K.B.E.

Other Members of Council.

Athol Lancelot Anderson, C.B.
David Anderson, LL.D., B.Sc.
George Ernest Bennett, M.Sc. (*India*).
Asa Binns.
Walter Miller Campbell (*South Africa*).
Raymond Carpmael, O.B.E.
Frederick Charles Cook, C.B., D.S.O., M.C.
Sir Harley Hugh Dalrymple-Hay.
Jonathan Roberts Davidson, C.M.G., M.Sc.
Charles George Du Cane, O.B.E., B.A.
Thomas Peirson Frank.
Ralph Freeman.
Griffith John Griffiths.
William Thomson Halcrow.
Charles George Hawes, B.Sc. (*India*).
Roger Gaskell Hetherington, C.B., O.B.E., M.A.
Ralph Frederick Hindmarsh.
Drummond Holderness (*New Zealand*).
Cecil Lee Howard Humphreys, T.D.
William Henry Morgan, D.S.O.
Sir Standen Leonard Pearce, C.B.E., D.Sc.

[1] The Council commence their term of office on the first Tuesday in November 1939.

Joseph Newell Reeson (*Australia*).
Vernon Alec Murray Robertson, M.C.
Francis Ernest Wentworth-Sheilds, O.B.E.
Julian Cleveland Smith, LL.D. (*Canada*).
Reginald Edward Stradling, C.B., M.C., D.Sc., Ph.D.

Mr. A. W. E. Bullmore proposed—That the thanks of The Institution be accorded to the Scrutineers, and that the ballot papers be destroyed. It needed no remarks of his, he said, to commend the resolution to the approval of the members.

Mr. P. J. Cowan seconded the motion, which was carried unanimously.

Mr. P. J. H. Unna said that all the Scrutineers would wish their thanks to be conveyed to the meeting. The ballot was considerably smaller than usual, over eight thousand members, including about two thousand five hundred overseas members to whom voting papers were not issued, having abstained from voting. He suggested that it might be a nice gesture to the overseas members if voting papers could be issued to all of them in future. He knew that that might be difficult owing to an obsolete by-law, which dated from before the War, and which had not visualized air mail services; but those who lived in the Malay States already received papers at the request of their Local Association, and he was sure that an invitation to take some direct part in the business of The Institution would be appreciated by all who lived abroad.

Presumably he should, in accordance with precedent, extend his thanks to twenty-three members who reduced the monotony of carrying out the scrutiny by disclosing the various ways in which it was possible to spoil a voting paper, and also to seven members who managed to spoil their voting papers by omitting to lick down the flap of the envelope, in order to save a penny stamp. He could never quite see why the flap should be licked down, and the only effective result on the present occasion was that one thousand five hundred and seventy-seven well-gummed envelopes had had to be cut open.

Mr. B. D. Richards proposed—That the thanks of The Institution be given to Mr. E. W. Monkhouse, Honorary Auditor, and that he be reappointed Honorary Auditor for the current financial year, and that Sir Allan Rae Smith be reappointed Professional Auditor for the current financial year.

Mr. A. S. Buckle seconded the resolution, which was carried unanimously.

Mr. J. S. Wilson asked if the moment were opportune to mention a point in connexion with the balloting paper. There were two overseas Members on the list in the present case whom he was glad to see had been elected. On the balloting paper it was stated that they were nominated as corporate members, resident in South Africa in one case and in Australia in the other. He did not know whether it was a fact, but he believed that

those members were nominated by other corporate members in South Africa and in Australia respectively. If on the balloting papers members were informed that those two Members had been nominated by their colleagues or by Local Associations in South Africa and Australia, the members would regard them far more favourably than if it were thought that they had simply been chosen without regard to the Local Association or to their colleagues in those countries.

Sir Clement Hindley said that at the President's request he would answer that question so far as he could. The Council, under the By-laws, were charged with the duty of preparing the ballot list, and it was their responsibility to put the names on the list; in regard to Members overseas, however, they received from the Advisory Committees in the localities certain suggested names, and from those suggestions the Council selected the names to be put on the balloting list. The Advisory Committees responsible for putting forward those suggestions were elected by the corporate members in the locality, and the Council were thus satisfied that the names put forward were acceptable to the general body of members. Although, therefore, it would not be right to say that the overseas Members in question were in any way voted for or balloted for separately by members overseas, it was known that they were people who were chosen and acceptable.

Mr. E. W. Monkhouse, on behalf of his fellow Auditor and himself, thanked the members for the resolution that had been passed.

Sir Cyril Kirkpatrick proposed—That the thanks of this meeting be accorded to Mr. William James Eames Binnie, President, for his conduct of the business as Chairman of the meeting. It required no words from him to say that Mr. Binnie had conducted the meeting in his usual business-like way and with his unfailing courtesy to everyone.

Mr. R. G. Hetherington seconded the resolution, which was carried unanimously.

The President briefly acknowledged the resolution.

Dr. W. L. Lowe-Brown said he was sure that all the members would wish to express their thanks to the Secretary and to his staff for the way in which they had done their work during the last year, which had been exceptionally busy owing to the central register which they had prepared in addition to their ordinary work.

Mr. H. G. Lloyd seconded the resolution, which was carried unanimously.

The Secretary, on behalf of the members of the staff and himself, thanked Dr. Lowe-Brown and Mr. Lloyd for their kind references to the Institution staff. At the same time he would like to take the opportunity of thanking the staff on his own behalf for their unfailing loyalty to himself.

The proceedings then ended.

REPORT OF THE COUNCIL, 1938–39.

In accordance with the By-laws, the Council present the following report upon the state of The Institution.

During the year under review the trend of international affairs has made it necessary to organize and accelerate measures for national defence, in which the services of the engineer play an ever-increasing part, and it may therefore be appropriate to deal briefly with some of the activities of The Institution in this respect.

National Service.—It will be recalled that early last year the Council decided to establish at the Institution a register of engineering services, in order that they might have readily available an up-to-date record of the qualifications and experience of corporate members for use as might be required in connexion with The Institution's work. At the time of the crisis in September last the compilation of the register was accelerated, and the appropriate Government department was informed that this register would be placed at their disposal if and when required for the utilization of the expert services of members in the event of emergency.

Subsequently the Government decided to organize the nation's man-power on the basis of voluntary registration, and the National Service Department of the Ministry of Labour was empowered to form a central register of scientists, technical experts, and professional men. The Institution is represented on the advisory council set up to advise on the utilization in war-time of persons on the central register, and the committee of that council dealing with the engineering section of the register is under the chairmanship of Mr. S. B. Donkin, Past-President. An invitation has recently been issued to all corporate members at home and in the Colonies to place their names on the central register, thereby offering their services to the Government if the need arose.

It may be noted that The Institution will continue to maintain its own record of engineering services for the purposes originally intended, and it is proposed to add to this record a list of firms of consulting engineers with their technical staffs.

Co-operation with Military Engineers.—Steps have also been taken to promote close co-operation between the military engineer and the civil engineer, both in times of peace and of war, and an informal meeting of The Institution held to discuss this question was attended by a large number of officers of the Corps of Royal Engineers. An informal liaison committee of civilian engineers and Royal Engineers has now been formed to foster and maintain closer co-operation, to create better opportunities for Royal Engineer officers to associate with their civilian brethren, and

to study how the best use can be made of the training and experience of such civilian engineers in the event of war.

The Institution having been legally advised that there is no provision in the Charters which would debar a Royal Engineer officer from election provided he can comply with the ordinary conditions for corporate membership, the Council have laid down the conditions under which such officers may be accepted; these conditions involve, in the case of officers commissioned since 1926 and who have passed the Mechanical Sciences Tripos Examination at Cambridge University, certain periods of training with civilian engineers in addition to their courses at the School of Military Engineering, Chatham, full compliance with the examination requirements being also a condition of their election. Officers commissioned prior to 1926 will be considered for corporate membership individually on their merits as regards adequate experience in civil engineering work and compliance with the examination requirements.

Engineering Precautions (Air Raid) Committee.—In May 1938 the Council, after consultation with the Air Raid Precautions Department of the Home Office and with the assurance of that department's co-operation, appointed a Committee to prepare and provide for engineers authoritative guidance regarding the protective measures necessary against damage by air bombardment to structures and other engineering works.

The Committee adopted a programme of research, and the major part of the experimental and theoretical investigation was undertaken by the Director of Building Research, Dr. R. E. Stradling, a member of the Committee. Facilities for this research work were provided by the Government, and The Institution allocated a member of the staff to act as technical officer to the Committee.

While the research investigation was being initiated the Committee decided to publish preliminary memoranda based on existing information. Two memoranda, on "Blast" and "Penetration of Bombs" respectively, have been published in the Journal and they have also been reprinted in pamphlet form.

The first stage of the research work has now been completed and a report has been prepared and presented to the Committee. It is hoped to publish this report.

The Committee have recently appointed a panel, upon which the Royal Institute of British Architects, the Institution of Mechanical Engineers, the Institution of Electrical Engineers, and the Institution of Structural Engineers have each nominated a representative, to collaborate with the Home Office in preparing design methods and type designs for protection of varying degree against the high-explosive bomb, based upon No. 5 Handbook of the Home Office.

Professional Assistance for the Government Scheme for Air Raid Shelters.—At the request of the Lord Privy Seal, The Institution is

collaborating with the Royal Institute of British Architects, the Institution of Structural Engineers, the Institution of Municipal and County Engineers, and the Chartered Surveyors' Institution, with a view to organizing the means of affording professional advice and assistance required by local authorities in carrying out the Government scheme for air raid shelters. For this purpose a committee representing the five Institutions, under the chairmanship of Sir Clement Hindley, Vice-President, known as the "Professional Advisory Committee (Shelters)", has been set up in London, together with regional professional committees in twelve provincial centres, and the local authorities have been requested by the Government to apply to these committees for the professional assistance required in carrying out air raid shelter schemes.

Institution Building.—The Council have given consideration to questions relating to the building and the safe custody of The Institution's records and property in case of national emergency.

Ordinary Meetings.—Mr. W. J. E. Binnie, M.A., delivered his Presidential Address at the opening meeting of the Session on the 1st November, 1938, when he departed from tradition and dealt with what was known by those who lived in ancient times regarding science and engineering. He gave numerous examples of ideas and in some cases of actual inventions which many years later came to fruition and were put into actual practice by modern scientists and engineers.

Eleven Ordinary Meetings have been held, at which the Papers mentioned below were discussed :—

SUBJECT.	AUTHOR.
The Principles of River-Training for Railway Bridges, and their Application to the Case of the Hardinge Bridge over the Lower Ganges at Sara.	Sir Robert R. Gales, F.C.H., M. Inst. C.E.
Improvements at the Royal Docks, Port of London Authority.	R. R. Liddell, M. Inst. C.E.
Strata Control in Coal Mines.	H. T. Foster, B.Eng., and M. A. Hogan, D.Sc., Ph.D., M. Inst. C.E.
The Conditions of Engineering Contracts.	E. J. Rimmer, M.Eng., B.Sc., Assoc. M. Inst. C.E., Barrister-at-Law.
The Gorge Dam.	W. J. E. Binnie, M.A., and H. J. F. Gourley, M.Eng., MM. Inst. C.E.
Some Experiments on the Lateral Oscillation of Railway Vehicles.	R. D. Davies, M.A., Ph.D., Assoc. M. Inst. C.E.
The Vertical Path of a Wheel Moving along a Railway Track.	Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S., M. Inst. C.E.

The Storstrøm Bridge.	G. A. Maunsell, B.Sc. (Eng.), M. Inst. C.E., and J. F. Pain, M.C., B.Sc. (Eng.), Assoc. M. Inst. C.E.
Considerations on Flow in Large Pipes, Conduits, Tunnels, Bends, and Siphons.	James Williamson, M. Inst. C.E.
Reconstruction of Aldgate East Station.	J. H. Harley-Mason, M. Inst. C.E.
Investigation of the Outer Approach Channels to the Port of Rangoon by Means of a Tidal Model.	Oscar Elsden, M.Sc., Assoc. M. Inst. C.E.
Schemes of Improvement for the Cheshire Dee: an Investigation by Means of Model-Experiments.	Jack Allen, M.Sc., Assoc. M. Inst. C.E.
The Singapore Airport.	R. L. Nunn, D.S.O., M. Inst. C.E.

The awards for Papers read and discussed at Ordinary Meetings, for other Papers published, and for Students' Papers, will be announced in the October Journal.

Supplementary Meeting:—

One supplementary meeting was held as follows:—

Subject.	Author.
The Problem of Stiffened Suspension Bridges, and its Treatment by Relaxation Methods.	R. J. Atkinson, B.E., and Professor R. V. Southwell, M.A., F.R.S.

Informal Meetings:—

Seven informal meetings were held as follows:—

Subject.	Introducer.
The Relative Merits of Pre-Cast and Cast-in-Situ Piles.	A. C. Dean, M.C., M.Sc., M. Inst. C.E.
The Design of Trunk Roads for High Speed and Safety.	H. E. Aldington, M. Inst. C.E.
Co-operation between the Civilian Engineer and the Military Engineer.	Sir Clement Hindley, K.C.I.E., M.A., M. Inst. C.E., and Lt.-Col. F. H. Budden, M.C., late R.E.
The Work of the Engineering Precautions (Air Raid) Committee.	Professor A. J. Sutton Pippard, M.B.E., D.Sc.
The Preparation and Giving of Expert Evidence.	J. H. Rider, M. Inst. C.E., M.I.E.E.
The Economic Aspect of the Replacing of Manual Power by Mechanical and Electrical Power.	Sir Henry Japp, K.B.E., M. Inst. C.E. (*since deceased*).

The Importance of a Training in Management for Engineers. (The last three meetings were held jointly with the Institutions of Mechanical and Electrical Engineers.)	T. G. Rose, M.I.Mech.E.

Lectures.—The Dugald Clerk Lecture on "Dugald Clerk and the Gas Engine—His Life and Work" was delivered before the Association of London Students on the 14th December, 1938, by Mr. W. A. Tookey, M. Inst. C.E., and was repeated at meetings of five of the Local Associations.

The forty-fifth James Forrest Lecture was delivered on the 2nd May by Professor Karl von Terzaghi, Dr.-Ing., M. Inst. C.E., who took for his subject "Soil Mechanics—A New chapter in Engineering Science."

Joint Meetings:—Three additional Joint Meetings were held with other Institutions and engineering bodies as follows :—

With the Institution of Structural Engineers, when a Lecture on "The Construction of Bridges in Denmark" was delivered by Professor Anker Engelund, M. Ing. F.

With the British Section, Société des Ingénieurs Civils de France, and the Institution of Structural Engineers, when a Paper on "The Strengthening of Austerlitz Bridge (Paris) by Electric-Arc Welding" was read by Monsieur Fauconnier.

With the Institution of Automobile Engineers and seventeen other bodies, when a symposium of Papers on "Factors Contributing to Comfort in Travel" was read and discussed :—

Section 1. "By Road", by S. E. Garcke, M.I.Mech.E., M. Inst. T.
Section 2. "By Rail", by the Rt. Hon. Lord Stamp, G.B.E.
Section 3. "By Air", by Captain E. W. Percival, M.R. Ae. S.

Road Engineering Section.—This Section was formed during the Session to include within its scope all matters appertaining to the study of road engineering relating to planning, construction, and research, and including highway bridges.

It was decided that the activities of the Section should include the following :—

(*a*) The holding of one Ordinary Meeting of The Institution, presided over by the President, to be devoted each Session to a Paper selected by the Section Committee and approved by the Council on the recommendation of the Publications and Library Committee.

(*b*) The holding of such number of Sectional Meetings as may be determined by the Section Committee, the choice of Papers, or of the subjects for informal discussion, for such Meetings

being in the hands of the Section Committee, subject to the approval of the Council.

(c) The arranging of a Summer or Autumn Meeting, if thought desirable, at which Papers can be read and discussed and visits to works organized, such arrangements being in the hands of the Section Committee, subject to the approval of the Council.

The opening meeting was held on the 8th February, 1939, when Mr. F. C. Cook, C.B., D.S.O., M.C., Member of Council, the Chairman, gave an Address on the aims and objects of the Section. A further meeting was held on the 30th March at which the subject of "Soil Problems in Road Construction" was introduced for discussion by Mr. A. H. D. Markwick, M.Sc., Assoc. M. Inst. C.E.; a ballot was also held for the election of the Committee of the Section. Over nineteen hundred Members, Associate Members, Associates, and Students have registered themselves as members of the Section.

Association of London Students.—The opening meeting of the Association of London Students was held on the 16th November, 1938, when Mr. A. L. Wheeler, B.Sc. (Eng.), Stud. Inst. C.E., Chairman of the Association, gave an Address in which he considered the aims and objects of the Association of London Students, and then dealt with some aspects of dock reconstruction, with particular reference to dredging, as carried out by the Port of London Authority. Mr. W. J. E. Binnie, President, took the Chair.

Three Meetings were held for the reading and discussion of Papers, and two Informal Meetings took place. One of the meetings was held jointly with the junior Sections of the Institutions of Mechanical and Electrical Engineers, and dealt with "The Engineering Aspects of the London Passenger Transport Board."

The fifty-eighth Annual Dinner of the Association was held at the Connaught Rooms on the 3rd March, when Mr. W. J. E. Binnie, President, was the guest of the evening. About seventy Students and guests were present.

Four Visits were paid to engineering works, but it is regretted that, with the exception of the visit to Waterloo Bridge, the attendance was less than usual.

Authority has been given for the publication on behalf of the Association of a special Students' Journal, of which Numbers appeared in October 1938 and January 1939 respectively. Reports received indicate that this has been much appreciated and should help to stimulate the interest of the Students in the work of the Association.

Engineering Abstracts.—The issue of "Engineering Abstracts" monthly in sectionalized form, each section dealing with one or with two or more allied branches of engineering, is being continued, and the

inclusion in the "Abstracts" of references to important articles appearing in journals published in the British Isles has been welcomed, especially by subscribers overseas.

"Ingenuity" Competition.—The Council have awarded the prize of £25 to Mr. W. E. Reed, Assoc. M. Inst. C.E., for his article submitted in competition under the will of Mr. R. E. S. Cooper, M. Inst. C.E., for the best answer to a problem on "Ingenuity", or the power to deal with an emergency cheaply, effectually, and quickly.

Institution Journal.—The attention of the Council has been drawn to the fact that it is not possible for those members who do not take the Institution Journal to keep in full touch with the various activities of The Institution, particulars of which are published in the Notices Section from time to time; and they strongly urge that every member should make a point of receiving the Journal, and interest himself in the different branches of work of The Institution.

Local Associations.—The Council having acceded to a request by the members in the Edinburgh district for the formation of a Local Association of Corporate Members and Students having its centre in that city, the inaugural meeting was held at Edinburgh on the 26th October, 1938. Professor Sir Thomas Hudson Beare, LL.D., F.R.S.E., M. Inst. C.E., was elected Chairman of the new Association, and the President (Mr. S. B. Donkin) attended the meeting and delivered a short address.

A request for the formation of an Association of Corporate Members and Students at Glasgow in place of the existing Students' Association has been acceded to, and it is expected that the new Association will be inaugurated at the beginning of next Session.

The Council wish these two Associations every success and they trust that their activities will be of much value to members and Students in Scotland.

The Council have also agreed to a proposal that Local Associations may, if they so desire, adopt a territorial nomenclature. As a result of this decision, the Manchester Association and the Portsmouth, Southampton and District Association have chosen to be named the North-Western Association and the Southern Association respectively, and the area covered by the latter Association has been considerably extended.

The Table on p. 156 gives in detail the strength and activities of the Local Associations.

In addition to the meetings included in the Table, a number of joint meetings have been held with the local branches of other engineering Institutions.

Papers have been read before the Local Associations on a great variety of subjects, including flood problems and land drainage; water-supply and sewage purification; slipways and wharves; aerodrome construction; electricity generation; welding; town planning; lighthouses and aids to navigation; and work in asphyxiating atmospheres. The Council are

Association.	Corporate members.		Students.	No. of meetings.	No. of visits.	Average attendance at meetings.
	Potential.	Actual.				
Birmingham	567	207	218	9	5	77
Bristol	212	98	100	10	—	38
Glasgow	—	—	171	7	4	33
Edinburgh	172	77	88	5	3	60
North-Western	645	154	164	11	3	42
Newcastle	294	125	93	17*	5	32
Northern Ireland	93	62	22	9	—	41
South Wales and Monmouthshire	182	57	62	6	1	45
Southern	438	81	128	9	3	58
Yorkshire	445	187	155	10	2	29

* Includes 5 meetings at Newcastle confined to Students only.

particularly pleased to record that a higher percentage of Papers than usual have been read by Students.

It is hoped that now that the general extension of boundaries has become operative corporate members will give a greater measure of support to the local activities.

With the exception of the Edinburgh and District Association, all of the Associations have held annual dinners during the past Session and these have been successful and well-attended functions.

Overseas Associations.—The Council have to record the formation of an Association of The Institution in the West Indies, and their best wishes are extended to this new branch. They trust that in spite of the long distances separating the members it will be possible to have some successful and well-attended meetings.

The Buenos Aires Association has 90 Corporate Members and 12 Students upon its roll—a slight decrease as compared with last year. During the Session six meetings have been held, at one of which Papers by three Students were submitted for discussion, and three visits have taken place, including one to Uruguay from the 20th to the 23rd October. The annual dinner of the Centre of British Engineering and Transport Institutions was held on the 7th October, the guest of honour being the British Ambassador, Sir Esmond Ovey, K.C.M.G., M.V.O. Awards of the Follett Holt premiums for 1937 have been made to Mr. D. G. MacCormack, B.Sc., M. Inst. C.E., and to Mr. D. H. Campbell, Stud. Inst. C.E.

The membership of the Malayan Association in October, 1938, was 120, compared with 117 for the previous year. Five meetings have been held and six visits were paid to engineering works. The annual dinner was held in Singapore on the 15th October, the guest of honour being the Chief Justice, the Hon. Mr. P. A. McElwaine, K.C. A premium of $100 has been awarded to Mr. J. J. Bryan, B.Sc., Assoc. M. Inst. C.E., for his Paper entitled " An Introduction to the Subject of Soil Mechanics."

The Shanghai Association has continued to hold meetings jointly with the local branches of the Institutions of Mechanical and Electrical Engineers and the Engineering Society of China. During the Session four Papers were read by members of the Association, and the annual dinner was held in Shanghai on the 24th February, when the Association was again honoured by the presence of H.M. Ambassador to China.

The report of the Victorian Association for the past Session has not yet been received; that for the year 1937–38 showed that there were then 34 Members, 48 Associate Members and 4 Students on the Roll of the Association, and that in addition to the annual general meeting, an annual dinner and two general meetings were held during the Session.

Nominations and Appointments.—The Council nominated Dr. David Anderson, B.Sc., to represent The Institution at the Celebration of the 300th Anniversary of the birth of Professor James Gregory, Professor of Mathematics in the University of St. Andrews, 1668–1674, which was held on 5 July, 1938; Mr. W. J. E. Binnie, M.A., to represent The Institution at the Engineering Congress in Wellington at the time of the Centenary Celebrations of the Foundation of New Zealand, to be held in 1940; and Mr. W. J. E. Binnie, M.A., and Sir Richard Redmayne, K.C.B., M.Sc., to represent The Institution at the 18th International Geological Congress, to be held in London from the 31st July to the 8th August, 1940.

Various other nominations and appointments have been made or renewed by the Council during the past year, and The Institution is or has been represented on advisory or administrative bodies and committees as detailed in Appendix I to this Report (pp. 166–167).

Corporate Members and Publicity.—In response to enquiries made as to what latitude may be allowed corporate members generally in connexion with publicity, the Council have issued the following three pronouncements:—

1. That it will not be considered to be a breach of the By-laws and Regulations governing Professional Conduct for a member to permit his name to appear with illustrations of works with which he has been professionally connected, published as parts of advertisements by contractors or manufacturers, but the member is expected to ensure that his name appears in an unobtrusive manner and not in any way as suggesting solicitation of professional work.

2. That there is no objection to a Corporate Member writing articles in the Press under his name describing a completed engineering work or one in course of construction for which he has been or is responsible, provided there is no solicitation of professional work contained in the article.

3. That the pronouncement of the Council, dated May 1934, be rescinded, namely:—

In response to inquirics the Council have ruled:

That while no objection would be taken to the appearance of

the names of Chartered Civil Engineers on memorial stones on completed works, it would be considered unprofessional for such names to be exhibited on works in course of construction, except where necessary for the purpose of direction, such as on the door of the resident engineer's office;

and that the following pronouncement be substituted:—

That there is no objection to the appearance of the names of Corporate Members on commemorative tablets and stones on completed works, and that a Corporate Member may exhibit his name on works in course of construction.

Research.—During the past year the conclusions reached by three committees have been published in Reports entitled "Recommendations in regard to Breathing Apparatus for Use in Sewers", "Regulations for Earthing Electrical Installations to Metal Water-Pipes and Water-Mains", and "Code of Practice for the Design and Construction of Reinforced-Concrete Structures for the Storage of Liquids." No further research on Breathing Apparatus for Use in Sewers or on the general aspects of reinforced-concrete liquid-retaining structures is contemplated at present, but, arising out of the latter subject, a new committee has been set up to study Bituminous Jointing Materials for Concrete. The Regulations for Earthing are subject to any amendment that may be shown to be desirable in the course of an extended programme of research on Earthing which is now in hand.

The researches on Vibrated Concrete, Pile-Driving, Fish-Passes, Repeated Stresses in Structural Elements, and Special Cements for Large Dams, are now nearing completion, but those on Wave-Pressures on Sea Structures, Simply Supported Steel Bridges, Velocity Formulas, and Earth Pressures, have been extended for further periods. The last-named research is receiving technical and financial support from the railway companies. A 10-year programme of research into the Soil Corrosion of Cement Products has been undertaken, and a similar programme for Metals is being drawn up.

Draft British Standard Specifications referred to The Institution for comment have continued to be examined by the Research Committee.

"Notes on Research Publications" are now issued monthly in duplicated form, available on application.

Sea-Action Committee.—The Seventeenth (Interim) Report of the Committee on the Deterioration of Structures exposed to Sea-Action has been published. The Eighteenth Report, which is now in preparation, will consist of a summary and critical examination by Dr. J. Newton Friend of the results of the 15-year tests on iron and steel specimens.

The Committee's investigations on the deterioration of timber have suffered a set-back, owing to the death of Professor George Barger, F.R.S., who had for many years carried out the preparation and examination of

timber specimens for the Committee. Arrangements are being made for the continuance of the work in hand.

Tests on reinforced-concrete specimens exposed at the Building Research Station, at Sheerness, and at the Gold Coast are being continued.

London Buildings Acts (Amendment) Bill, 1938–39.—On the report of a Special Committee appointed by the Council to examine the provisions of this Bill, the Council decided that there were not sufficient grounds, either from an engineering or other point of view, to warrant The Institution petitioning against any of its provisions.

L.C.C. Building By-Laws.—The Joint Committee of The Institution, the Royal Institute of British Architects, the Chartered Surveyors' Institution, and the Institution of Structural Engineers, which was appointed to examine the London County Council Building By-laws, has, at the request of the County Council, considered draft regulations relating to the following :—the use of structural steel other than structural steel as required and provided for in the Building By-laws ; the use of fire protection for structural steelwork other than that required by the Building By-laws ; and the use of oxy-acetylene welding instead of riveting, bolting, or lapping. A number of suggestions on these subjects have been communicated to the County Council.

The Committee wish it to be known that they will be glad to have brought to their notice any cases of hardship arising under the new Building By-laws, in order that they may, if considered desirable, make representations to the County Council on the subject when the By-laws are reviewed in 1941.

James Alfred Ewing Medal.—The award of the James Alfred Ewing Medal for 1939 for specially meritorious contributions in the field of engineering research has been made to Professor Arnold Hartley Gibson, D.Sc., M. Inst. C.E., on the recommendation of the President and Sir William Bragg, President of the Royal Society, following suggestions put forward in their personal capacities by the Presidents of the Institutions of Mechanical and Electrical Engineers and Naval Architects. The presentation was made by Mr. Binnie on the 2nd May.

Scholarships.—William Lindley Scholarships of the value of £80 per annum for 3 years have been awarded to Mr. James Harvey Trevithick Ivory and Mr. Percy Gordon Spencer to assist them to pursue courses of engineering study at the University of Cambridge and at King's College, London (or one of the Northern Universities) respectively ; and a William Lindley Scholarship of £80 for 1 year has been awarded to Mr. Robert Haigh Walker, Stud. Inst. C.E., to assist him to complete his final year's course of engineering study at King's College, London.

A Dennison Scholarship of £20 per annum for 3 years has been awarded to Mr. Richard Gordon Nelson, Stud. Inst. C.E., to assist him to pursue a course of engineering study at the University of Glasgow.

Charles Hawksley Prize.—On the report of the judges (the President

of The Institution and the President of the Royal Institute of British Architects), the Council have awarded a Charles Hawksley Prize of £150 to Mr. Donald Henry May, Assoc. M. Inst. C.E., for his design of a grain silo, and have honourably mentioned Mr. Arthur James Francis, M.Sc., Stud. Inst. C.E., for the merit displayed in his design of a steel highway bridge.

Conditions of Contract.—At the request of the Federation of Civil Engineering Contractors, the Council have given attention to the practicability of setting up standard conditions of contract for engineering works, and a small committee representative of The Institution, the Federation and the Association of Consulting Engineers, under the Chairmanship of the President of The Institution, has been formed to consider this matter.

International Engineering Congress, Glasgow.—An International Engineering Congress was held at Glasgow from the 21st to the 24th June, 1938, in which The Institution took part with a number of other engineering societies. The Institution was represented at the meetings of the Congress by the President (Mr. Donkin) and the Secretary.

Annual Dinner and Conversazione.—The Conversazione took place on the evening of the 15th June, 1938, when some 2,750 members and ladies were present; the Annual Dinner was held on the 23rd March, when 657 members and guests attended.

Forthcoming Events.—In August last an invitation was received from the American Society of Civil Engineers to participate with that Institution and the Engineering Institute of Canada in a joint meeting to be held in New York in September, 1939, during the New York World's Fair. The Council had much pleasure in accepting this invitation on behalf of The Institution, and an interesting programme of meetings and visits is being arranged. At the conclusion of the New York meeting a visit will be paid to Washington, followed by a brief tour of eastern Canada, the party returning from Montreal or Quebec as preferred. It is hoped that as many members as possible will take this opportunity to meet American and Canadian engineers and to discuss problems of mutual interest.

A Summer Meeting of The Institution will be held at Birmingham from the 6th to the 8th July next. The meeting, which is being organized in conjunction with the Committee of the Birmingham and District Association, will be the first meeting of The Institution to be held in that city. On the 7th July there will be a short technical session, at which Papers on "Town and Country Planning" by Mr. H. J. Manzoni, M. Inst. C.E., and on "Sewerage and Sewage-Disposal" by Mr. John D. Watson, Past-President Inst. C.E., will be read, and on the evening of that day there will be a civic reception at the Council House by the Lord Mayor of Birmingham. The remainder of the meeting will be devoted to visits to works and places of interest in the neighbourhood, including a visit on the 8th July to the new airport at Elmdon to be officially opened on that day by H.R.H. the

Duchess of Kent. The presence of ladies at the meeting will be welcomed and a good attendance of members and Students is anticipated.

The Council will promote a conference on Engineering Education and Training during the course of next Session. An invitation has been issued to universities and bodies interested in this subject to appoint representatives on a general committee to organize the conference.

Accounts.—The Accounts for the year ending 31st March, 1939, which have been duly audited, are detailed in Appendix II of this Report (pp. 168 *et seq.*), and may be summarized briefly as follows :—

The *Total Income* for the year amounted to . . (as compared with £44,056 last year) including £346 for Income Tax recovered. Subscriptions, Entrance and Examinations fees totalled £45,336 (as compared with £42,296 last year) and Dividends and Interest received amounted to £1,208 (as compared with £1,390 last year).	£47,117
The *Total Expenditure* charged against the year's Income amounted to . . . . (as compared with £45,718 last year) including Provisions of £14,600 (viz. £12,000 for Publications Account and £2,600 for Research Reserve).	£45,787
The *Revenue Account* therefore results in a credit balance of . . . . from which has been deducted the debit balance of £1,049 brought forward from last year, resulting in a final credit balance of £281 as shown on the Balance Sheet.	£1,330

The actual expenditure during the year on "Publications Account" amounted to £16,295 (compared with £19,493 last year), of which £10,704 represented the cost of the Journal. This expenditure was relieved by credits for advertisements, sales, etc., of £5,182 (against £4,650 last year), leaving the net expenditure for the year at £11,113 (compared with £14,843). This amount was £887 less than the £12,000 provision, and the overspent balance of £8,923 brought forward from last year has thus been reduced to £8,036. The latter figure falls to be liquidated in the future.

The *Research Reserve* credit balance has increased by £1,097 during the year, viz. from £1,865 to £2,962. The expenditure incurred amounted to £3,063 whereas the credits (made up of the appropriation from Revenue Account of £2,600, and contributions by outside bodies of £1,560) totalled £4,160.

The Repairs and Renewals Reserve credit balance has been reduced by £5 during the year, viz. from £5,216 to £5,211.

On Trust Funds Income Account there was received a total of £1,241 (£1,323) and the expenditure amounted to £1,048 (£996).

Contributions amounting to £25 (£627) were received from Home and Overseas Harbour and Dock Authorities towards the cost of the research into the Deterioration of Structures exposed to Sea Action. (The reduction in the year's contributions is due to the termination of the period for which the yearly subscriptions were originally granted.) The expenditure during the year was £691 (£537).

Library.—During the year 865 volumes were presented to the Library and 188 were purchased, making a total, on the 31st March, 1939, of 64,182.

The number of applications received for books on loan was 2,143, an increase of 101 on the previous year. At the end of 1938, a Supplement was published to the 1935 Loan Library Catalogue, and there is a steady demand for this and for the original Catalogue.

Opportunity was taken during the Session to clear the Main Library shelves, which were becoming congested, and in the result some 2,000 volumes were transferred to new shelving in the Basement and Lower Ground Floor book stores. The Council authorized the removal, from the Library, of some 2,000 items which were not of engineering interest, and these have been duly deleted from the Catalogue.

For the benefit of members wishing to have access to the joint library of the Iron and Steel Institute and the Institute of Metals, an interchange of library facilities has been arranged between The Institution and these bodies.

Gifts.—Mr. S. B. Donkin, President 1937–38, has presented to The Institution his portrait, by Francis Dodd, R.A. The Council have also accepted a framed Certificate of Membership of J. G. Lynde, dated 1844, grandfather of the late G. G. Lynde, Member, which had been presented by Mrs. Lynde, and two rare pamphlets by John Smeaton, F.R.S., presented by Mr. W. T. Halcrow, Member.

Examinations.—The number of candidates presenting themselves for the October, 1938, examinations was 457, namely, 66 for the Preliminary Examination and 391 for the Associate Membership Examination. The entries for the April, 1939, examinations were 137 for the Preliminary Examination and 644 for the Associate Membership Examination.

Bayliss Prizes of the value of £15 have been awarded to Mr. Eric Cardale Whitaker, Assoc. M. Inst. C.E., and to Mr. Colin Bruce Hornig, Stud. Inst. C.E., in respect of Sections A and B of the Associate Membership Examination for April and October, 1938, respectively, and Mr. Kuppuswami Subrahmanyan, B.E., Assoc. M. Inst. C.E., has received honourable mention in connexion with the former examination.

Elections, Transfers, and Admissions.—The number of candidates has been again very large. 542 new Proposals for Election have been received during the year and 162 postponed from previous Sessions—mainly pending compliance by the candidates with the Examination requirements—were brought forward for final consideration, making a

total of 704 applications dealt with by the Council. 112 recommendations for the Transfer of Associate Members to the class of Members have also been considered and 566 new Students were admitted.

The Roll.—The Roll of The Institution on the 31st March, 1939, stood at 12,613, the changes which took place during the year ended on that date being shown in the following Table :—

	1 April, 1937, to 31 March, 1938.						1 April, 1938, to 31 March, 1939.					
	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.	Honorary Members.	Members.	Associate Members.	Associates.	Students.	Totals.
Numbers at commencement .	17	2288	7199	57	2187	11,748	19	2286	7349	54	2433	12,141
Transfers—Associate Members to Members	..	85	85	..	..		..	101	101	..	..	
Elections .	4	7	337	3	..		..	17	408	3	..	
Admissions .	..	..	..	..	572	+940	..	..	..	..	581	+ 1,023
Restored to Roll .	..	2	14	..	1		..	7	6	..	1	
Deceased .	2	64	61	3	3		..	65	59	..	4	
Resigned .	..	26	29	2	9		..	21	45	2	9	
Erased .	..	5	23	1	17		..	2	13	1	16	
Elected as an Honorary Member .	..	1	..	..	..		..	..	..	..	..	
Elected as Associate Members	..	..	..	..	177	— 547	..	..	..	..	193	— 551
Removed—over age .	..	..	..	..	116		..	..	..	..	115	
Failed to complete .	..	..	3	..	3		..	1	2	..	3	
Failed to comply (Studentship)	..	..	..	..	2	+ 393	..	..	..	..	..	+ 472
Numbers at termination	19	2286	7349	54	2433	12,141	19	2322	7543	54	2675	12,613

The Roll at this date is 12,664.

During the year the elections comprised 17 Members, 408 Associate Members, and 3 Associates; 581 candidates were admitted as Students, and the names of 7 Members, 6 Associate Members, and 1 Student were restored to the Roll. From this addition of 1,023 must be deducted the deaths, resignations, erasures, over-age Students, and the Students elected Associate Members, amounting to 551 in all, showing a net increase of 472; 101 Associate Members have been transferred to the class of full Members.

The Council record with especial regret the deaths of Sir John Purser Griffith, M.A.I., and Sir Basil Mott, Bart., C.B., F.R.S., Past-Presidents; Alexander Newlands, C.B.E., Member of Council; Sir John Francis Cleverton Snell, G.B.E., former Vice-President; Sir Albert Ashley Biggs; Claude Charles Carpenter, C.B.E., D.Sc., Sir Henry Fowler, K.B.E., D.Sc., LL.D., and Edward John Way, former Members of Council.

The full list of deaths is as follows (*E.* refers to election, *T* to transfer and *A.* to admission) :—

DEATHS.—*Members* (65).—Frederick Adams (E. 1892. T. 1903); George Samuel Burt Andrews (E. 1894. T. 1905); John Ashurst (E. 1936); Arthur Charles Auden (E. 1891. T. 1913); Howard Walter Basden-Smith (E. 1920); *Sir* Albert Ashley Biggs (E. 1900. T. 1910) (*former Member of Council*); Frederick Bluett (E. 1887. T. 1903); Hubert Francis Toriano Bode (E. 1893. T. 1916); Edward Box (E. 1896. T. 1908); *Col.* Francis Boynton, R.E. (E. 1884. T. 1893); *Professor* William Hubert Burr (E. 1893); Edward Kynaston Burstal (E. 1876. T. 1881); Charles Claude Carpenter, C.B.E., D.Sc. (E. 1886. T. 1896) (*former Member of Council*); Herbert Turlay Chapman (E. 1918); Henry William Clothier (E. 1898. T. 1937); John Cormack Craig, O.B.E., D.S.O. (E. 1935); *Sir* Philip Dawson, M.P. (E. 1894. T. 1904); Lawrence Ennis, C.M.G., O.B.E. (E. 1933); Edmund Alderson Sandford Fawcett, C.B. (E. 1893. T. 1904); John Ferguson (E. 1892. T. 1902); Alfred Fidler (E. 1893. T. 1905); *Sir* Henry Fowler, K.B.E., D.Sc., LL.D. (E. 1896. T. 1918) (*former Member of Council*); John Gass (E. 1904); William Gilbert (E. 1893. T. 1903); Riyosaku Godai (E. 1885. T. 1920); Charles Henry Godfrey (E. 1899. T. 1914); George Abraham Goodwin (E. 1880. T. 1893); *Sir* John Purser Griffith, M.A.I. (E. 1877. T. 1883) (*Past-President*); Geoffrey Harnett Harrisson, C.M.G., D.S.O. (E. 1908. T. 1918); Vincent Hart, C.S.I. (E. 1907. T. 1936); David Hay (E. 1892. T. 1895); Henry James Shedlock Heather, B.A. (E. 1892. T. 1905); *Hon.* Philip Henderson (E. 1911. T. 1919); David William Herbert (E. 1893. T. 1905); Charles Hugh Higson (E. 1901); Henry George Humby (E. 1881. T. 1891); James Howden Hume (E. 1930); *Sir* Herbert Henry Humphries, C.B.E. (E. 1920); Archibald Jack, C.B., C.M.G., C.B.E. (E. 1920); Maurice Harry King, B.Sc. (E. 1920. T. 1929); Henry Angley Lewis-Dale, C.B.E. (E. 1902. T. 1920); *Professor* Henry Louis, M.A. (E. 1907); Gerald Gascoigne Lynde (E. 1899. T. 1914); Robert Macalister (E. 1881. T. 1896); William Thomas Bartholomew McCormack (E. 1920); Alexander Pile Macdermid (E. 1919); William Morris Mordey (E. 1904); *Sir* Basil Mott, *Bart*,. C.B., F.R.S. (E. 1895) (*Past-President*); Alexander Newlands, C.B.E. (E. 1895. T. 1912) (*Member of Council*); James Just Niven (E. 1902. T. 1914); John Henry Parkin (E. 1889. T. 1918); Evan Parry, B.Sc. (E. 1910. T. 1919); Donald Paterson (E. 1910. T. 1925); Charles Blois Pooley, C.I.E. (E. 1931); Henry Augustus Ratcliff (E. 1933); Robert Ridgway (E. 1922); Henry John Rofe, M.A. (E. 1908); John Severs (E. 1913. T. 1919); Percy Philip Oscar Smith (E. 1908. T. 1927); *Sir* John Francis Cleverton Snell, G.B.E. (E. 1895. T. 1903) (*former Vice-President*); David Alan Stevenson, B.Sc., F.R.S.E. (E. 1884. T. 1886); John Alfred Vaughan (E. 1908); Edward John Way (E. 1901. T. 1908) (*former Member of Council*); *Sir* George Whitehouse, K.C.B. (E. 1882. T. 1892); George Beck Wilkinson (E. 1905).

Associate Members (59).—Matthew Atkinson Adam, B.Sc. (E. 1900); Horace Allen (E. 1884); George William Anderson (E. 1892); Arthur Alan Ashworth, M.A. (E. 1893); Ronald Baird, B.Sc. (E. 1934); Charles John Seymour Baker (E. 1881); Percy Baylis (E. 1884); John Morton Billing, B.Sc. (E. 1928); Richard Francis Ponsonby Blennerhassett (E. 1906); William Boby (E. 1889); Charles Botterill (E. 1896); William Dugald Camphell (E. 1875); Henry Joseph Caple (E. 1924); Jesse Fairfield Carpenter (E. 1889); Reginald Sorré Cole, M.A. (E. 1901); James William Daniel Cook (E. 1928); Charles Edward Davenport (E. 1890); Joseph Draper (E. 1908); John Dobie Halliday Dymock, B.Sc. (E. 1911); Sydney Firth (E. 1883); Ebenezer Fraser (E. 1905); Howard Goodfellow (E. 1907); Francis Green (E. 1924); Carl Thomas Alfred Hanssen (E. 1900); Charles Nelson Hefford, M.Sc. (E. 1900); William Halcot Hingston (E. 1909); Arthur Fayrer Hosken (E. 1895); Ronald Inglis (E. 1931); Edward Henry James (E. 1913); Ernest Katzenstein (E. 1900); Leonard

Leeper, B.E. (E. 1908); Duncan Mackenzie Mactavish, B.Sc. (E. 1931); *The Right Hon. Lord* Marks of Woolwich, C.B.E. (E. 1884); James Meldrun (E. 1897); Joseph Mulgrew (E. 1928); Birra Venkatareddy Narayana Rao (E. 1932); Florence O'Driscoll (E. 1887); Thomas Partington (E. 1885); Arthur Peirce (E. 1896); Henry Vivian Majendie Phelips (E. 1886); William Marshall Philip (E. 1888); Sidney John Powell (E. 1900); William Edward Price (E. 1891); Thomas Noble Ritson (E. 1885); Frank Round (E. 1910); William Colin Russell (E. 1908); George Minard Seels (E. 1912); Moharrem Sabet Sidahmed, B.Sc. (E. 1924); Walter Strapp (E. 1885); Sidney Mark George Teal (E. 1915); Arthur Noel Thorpe (E. 1904); William Henry Todd (E. 1890); John Herbert Veasey (E. 1921); Robert Arthur Waddell (E. 1915); James Arthur Watts (E. 1905); Ernest Frederick Welch (E. 1880); *Sir* Robert Williams, *Bart.* (E. 1887); Dudley Yorke (E. 1928); Jules Dent Young (E. 1882).

Students (4).—Clifford Albert Burmister (A. 1937); Norman Herbert Dick, B.Sc. (A. 1935); Donald Crane Howard (A. 1938); Stewart Webb, B.Sc. (A. 1932).

The following resignations have been received:—

Members (21).—William Percy Appleford (E. 1925); Frederick Grant Brighton (E. 1904. T. 1924); Thomas Edward Burrows (E. 1902) (*since reinstated*); Henry Ralph Crabb (E. 1905. T. 1923); *Sir* John Egan Eaglesome, K.C.M.G. (E. 1910); Francis Douglas Fox, M.A. (E. 1893. T. 1902); Patrick Henry Holland, B.E. (E. 1915. T. 1933); George Kermode, M.C.E. (E. 1899. T. 1913); Richard Priestley Lawrence (E. 1899. T. 1909); Robert William Menmuir (E. 1899. T. 1916); Philip Benjamin Osborn (E. 1911); Harold Gordon Peake, B.Sc. (E. 1916. T. 1931); Herbert James Bingham Powell, O.B.E. (E. 1904. T. 1914); William Murray Pullar, B.C.E. (E. 1914. T. 1926); Herbert Martin Setchell (E. 1925); George Malcolm Roslin Sinclair (E. 1925); Henry Joseph Trivess Smith, C.B.E. (E. 1907. T. 1913); George Sturrock (E. 1907. T. 1932); Lionel Percy Duncuft Taylor, O.B.E. (E. 1911) (*since reinstated*); Edward Gentry Timbrell, B.Sc. (E. 1921. T. 1934); Richard Fitzarthur Waller (E. 1896. T. 1927).

Associate Members (45).—James Armitage, M.A. (E. 1929); James Arthur Ernest Ball (E. 1919); Robert Brown (E. 1906); Harry Clegg (E. 1900); Ivor Parry Davies (E. 1913); Alfred Ernest Drown (E. 1903); Marmaduke Stephen Duffitt (E. 1912); Eric William Eller (E. 1918); Samuel Mark Folk (E. 1904); Stephen Glennie (E. 1912); Henry Herman Gordon, B.A. (1914); Richard Frederick Hartley, B.Sc. (E. 1904); Harold Bryning Hewlett, B.Sc. (E. 1911); Pao-chang Ho, B.Sc. (E. 1916); Percy Nicholas Hooper (E. 1899); Rutherford Vincent Theodore Hoskings (E. 1929); George David Lafayette Hunter (E. 1917); John Liversidge Jeffree (E. 1929); John Norman Lapage (E. 1908); George Bertram Leach (E. 1913); Charles Fergusson Nash Leahy (E. 1912); Christopher Henry Linnell (E. 1911); Arthur Burrow Linscott (E. 1902); Harry Reep Lintern, M.Sc. (E. 1927); Duncan Alexander MacDougall (E. 1905); John Rowland Jones McLean (E. 1901); John McNair (E. 1919); Arthur Ernest Mayes (E. 1893); *Sir* James Milne, K.C.V.O., C.S.I., B.Sc. (E. 1908); Cyril Pearce (E. 1924); William Pearce (E. 1916); George Edward Luther Poulden (E. 1897); Ernest Cecil Pound, B.Sc. (E. 1931); William Frederick Roger Reynolds (E. 1917); Wilfred John Robinson, B.Sc. (E. 1932); Douglas Albert Neville Sandifer, B.Sc. (E. 1928); Chauncey Hugh Sumner (E. 1907); Cyril Douglas Kidd Sutton, B.Sc. (E. 1931); John Edward Oakes Thompson (E. 1894); John Sherwood Todd (E. 1906); Robert Alfred Weiss (E. 1921); Bertram Arthur Whittle (E. 1909); William Garrick Wishart, B.Sc. (E. 1920); Reginald Wissenden (E. 1892); Frederick George Wolstenholme (E. 1906).

Associates (2).—*Brevet Major* Francis Vivian Lister, O.B.E. (E. 1911); *Rai Bahadur* Jagmal Raja (E. 1931).

Students (9).—Michael Vivian Duncan Braine (A. 1936); Francis Stanley Burt (A. 1937); Denis Frank Day (A. 1932); Shankar Vasudeo Desai, B.E. (A. 1933); Arthur Stanley Hall, B.Sc. (A. 1937); William Robert Cameron Houston (A. 1937); Sydney Cecil Hoxey (A. 1932); John Leonard Mitchell Ransford, B.Sc. (A. 1936); Gerard Francis Young (A. 1928).

APPENDIX I

Royal Commission for Exhibition of 1851	The President.
Grant Committee of the Royal Society	The President.
General Board of National Physical Laboratory	Sir Alexander Gibb, G.B.E., C.B., LL.D., F.R.S. Sir Richard A. S. Redmayne, K.C.B., M.Sc.
Department of Scientific and Industrial Research :—	
Committee on Testing Work for the Building Industry	The President (or his Deputy).
Mechanisation Board, Army Council	W. G. Wilson, C.M.G., B.A.
Advisory Panel on Transport (Ministry of Transport)	O. R. H. Bury. Col. R. E. B. Crompton, C.B., F.R.S. J. A. Saner.
Home Office Sub-Committee on Air Raid Precautions	Sir Leopold H. Savile, K.C.B.
Science Museum Advisory Council, Board of Education	Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S.
Engineering Joint Examinations Board	Professor C. E. Inglis, O.B.E., M.A., LL.D., F.R.S. R. E. Stradling, C.B., M.C., D.Sc., Ph.D.
Court of the University of Bristol	Raymond Carpmael, O.B.E.
Court of the University of Liverpool	Thomas Molyneux, O.B.E.
Court of the University of Sheffield	Sir William H. Ellis, G.B.E., D.Eng.
Governing Body of the Imperial College of Science and Technology	Sir George W. Humphreys, K.B.E.
Council of the City and Guilds of London Institute	The President.
City and Guilds Fellowship Selection Committee	E. G. Walker, B.Sc.
Court of University College, Southampton	F. E. Wentworth-Sheilds, O.B.E. A. C. Hughes, B.Sc.
Governing Body of the School of Metalliferous Mining, Cornwall	J. G. Lawn, C.B.E.
Thomason College, Roorkee, Advisory Council	Gerald Lacey, B.Sc.
Old Centralians Committee on Memorial to Dr. W. C. Unwin	Sir Richard A. S. Redmayne, K.C.B., M.Sc. Sir Charles L. Morgan, C.B.E., D.Eng. The Secretary.
Engineering Joint Council	David Anderson, LL.D., B.Sc. W. T. Halcrow.
Engineering Public Relations Committee	Sir Clement D. M. Hindley, K.C.I.E., M.A. Sir Leopold H. Savile, K.C.B. (Deputy).
Diesel Engine Users' Association	W. A. Tookey. H. R. Ricardo, B.A., F.R.S.
Council of the London Society	T. H. Bailey.
Governing Body of the Denning Trust	A. E. Cornewall-Walker.
Tribunal of Appeal, London Building Act, 1930	Sir Cyril R. S. Kirkpatrick.
Joint Committee on Materials and their Testing	R. H. H. Stanger.
Research Committee on High Duty Cast Irons of the Institution of Mechanical Engineers	Professor Gilbert Cook.
Welding Research Committee	Ralph Freeman.
Alloys and Iron Research Committee of the Institution of Mechanical Engineers	Sir Robert A. Hadfield, Bt., D.Sc., D.Met., F.R.S.

British Cast Iron Research Association	Sir Robert A. Hadfield, Bt., D.Sc., D.Met., F.R.S.
Permanent Commission of International Navigation Congresses	Sir Cyril R. S. Kirkpatrick. N. G. Gedye, O.B.E., B.Sc.
General Organizing Committee, 18th International Geological Congress, 1940	Sir Richard A. S. Redmayne, K.C.B., M.Sc. The President or his Deputy (*ex officio*).
Joint Committee of The Institution and the Institution of Structural Engineers on Code of Practice for Structural Steelwork	Professor J. F. Baker, M.A., D.Sc. Professor C. Batho, D.Sc., B.Eng. H. P. Budgen, Ph.D., M.Sc. Ralph Freeman. B. L. Hurst. Professor A. J. S. Pippard, M.B.E., D.Sc. J. D. Vaughan, M.Sc.
Joint Committee on Co-operation between Overseas Members of E.J.C. Institutions	Sir Clement D. M. Hindley, K.C.I.E., M.A.
Association of Scientific Libraries and Information Bureaux	C. G. Du Cane, O.B.E., B.A., with Lt.-Col. F. H. Budden, M.C., as Deputy.
National Service: Central Register, Advisory Council	S. B. Donkin.
National Service: Central Register, General Engineering Committee	S. B. Donkin. The Secretary.
World Power Conference British National Committee	S. B. Donkin.
World Power Conference International Sub-Committee on Special Cements	W. T. Halcrow.
International Electrotechnical Commission on Steam Turbines	I. V. Robinson.
International Electrotechnical Commission Advisory Committee on Internal-Combustion Engines	W. A. Tookey.

The Institution is represented as follows on Councils of the British Standards Institution :—

General Council	Sir Cyril R. S. Kirkpatrick.
Engineering Divisional Council	Sir Clement D. M. Hindley, K.C.I.E., M.A. W. T. Halcrow. J. R. Davidson, C.M.G., M.Sc.

and has also representatives on numerous Committees, Sub-Committees, and Panels.

Main Committee of the Canadian Engineering Standards Association	H. H. Vaughan.

APPEN

BALANCE SHEET

	£	s.	d.	£	s.	d.
To Institution Capital Account and Building Fund, *as per last account*	..			416,323	16	1
,, Creditors	..			3,551	11	0
,, Repairs and Renewals Reserve, *as detailed on pages 170 and 171*	..			5,211	15	10
,, Research Reserve, *as detailed on pages 176 and 177*	..			2,962	10	3
,, W. A. P. Tait Legacy, per last Account	173	3	7			
Less Expenditure on Memorial Panel	75	0	0			
				98	3	7
,, Sea-Action Committee Account, *as detailed on pages 176 and 177*	..			1,391	7	1
,, Trust Funds, Capital and Income Accounts—						
Capital Accounts, *as detailed on pages 174 and 175, invested per contra*	38,017	6	3			
Income Accounts—Balances unexpended—*as detailed on pages 176 and 177*	2,959	15	10			
				40,977	2	1
,, Institution Revenue in Suspense—						
Proportion of 1939 Subscriptions received applicable to the nine months from 1st April to 31st December, 1939	15,526	6	10			
Subscriptions received in advance	94	13	7			
				15,621	0	5
				£486,137	6	4

AUDITOR

We have audited the above Balance Sheet dated 31st March, 1939, and have obtain is properly drawn up so as to exhibit a true and correct view of the state of The Institutio shown by the books of The Institution.

London, 28th April, 1939.

DIX II.

31st MARCH, 1939.

			£ s. d.	£ s. d.
By Expenditure on Institution Building, including Cost of Site, *as per last account*				375,767 16 10
By Institution Investments (including those held in respect of Repairs and Renewals Reserve) at or under cost, *as detailed on page 178*				50,294 0 4
Note.—*The value of these Investments at ruling prices on 31st March, 1939, amounted approximately to £42,998.*				
,, W. A. P. Tait Legacy—				
Cash at Bank				98 3 7
,, Sea-Action Committee Account—				
Cash at Bank			230 19 1	
£1,200 3½% War Stock at cost			1,160 8 0	
				1,391 7 1
,, Trust Funds Investments, etc.—				
Capital :—				
Investments, *as detailed on pages 174 and 175*		38,015 4 3		
Cash at Bank		2 2 0		
			38,017 6 3	
Unexpended Income :—				
Investments, *as detailed on page 175*		214 8 2		
Cash at Bank—				
On Deposit a/c . .	2,500 0 0			
,, Current a/c . .	245 7 8			
		2,745 7 8		
			2,959 15 10	
				40,977 2 1
,, Debtors				1,777 19 2
Cash at Bank and in Hand—				
Current Accounts			7,941 12 1	
In Hand			134 12 11	
				8,076 5 0
,, Revenue Account—				
Publications Revenue Account—				
Balance overspent to date, *per page 170* . . .			8,036 2 2	
Less General Revenue Account surplus, viz., Surplus for year to date per account .		£1,330 2 0		
Less Deficit at 31st March, 1938		1,048 12 1		
			281 9 11	
				7,754 12 3
				£486,137 6 4

E. Graham Clark, *Secretary.*

PORT.

the information and explanations we have required. In our opinion such Balance Sheet
irs according to the best of our information and the explanations given to us, and as

Alan Rae Smith
E. W. Monkhouse, M. Inst. C.E. } Auditors.

RESERVE FOR REPAIRS AND RENEWALS TO

	£ s. d.
To Net Expenditure during the Year	1,199 2 1
Balance *carried down*	5,211 15 10
	£6,410 17 11

PUBLICATIONS

	£ s. d.	£ s. d.
To Balance, *per last account*		8,922 15 0
,, Expenditure during the Year—		
Journal	10,703 12 9	
Students' Journal	83 0 5	
Subject Index	104 3 0	
Charters, By-laws and Lists of Members	721 8 6	
Engineering Abstracts	1,064 16 10	
Reports, etc.	104 14 4	
Salaries, Clerical Pay and Pensions Premium	3,402 16 9	
Lecture	26 5 0	
Reporting	84 19 8	
	16,295 17 3	
Less Credits for Advertisements, Sales, Contributions, etc.	5,182 10 1	
		11,113 7 2
		£20,036 2 2
To Balance *brought down, as per Balance Sheet, page 169*		£8,036 2 2

STRUCTURE, FURNITURE, FITTINGS AND MACHINERY.

	£	s.	d.
By Balance, *per last account*	5,216	10	8
,, Institution Revenue Account—Amount provided for the year—*per page 172*	1,000	0	0
,, Interest on Investments	178	14	3
,, Income Tax Refunded	15	13	0
	£6,410	17	11
By Balance *brought down, as per Balance Sheet, page 168*	£5,211	15	10

REVENUE ACCOUNT.

	£	s.	d.
By Institution Revenue Account—Amount provided for the year—*per page 172*	12,000	0	0
,, Balance *carried down* (being Excess of net Expenditure over Provision)	8,036	2	2
	£20,036	2	2

GENERAL REVENUE ACCOU

EXPENDITURE.

1937-38 £		£	s.	d.	£	s
	To HOUSE AND ESTABLISHMENT CHARGES—					
	Rates, Health, Unemployment and other Insurances	6,462	8	3		
	Electric Lighting and Power, Water-Supply, Warming, Ventilating and Telephone	723	14	9		
	Cleaning and Household Expenses	888	18	2		
	Refreshments and Assistance at Meetings	102	8	7		
7,979					8,177	9
	,, REPAIRS AND RENEWALS RESERVE—					
1,000	Amount provided for the year, *per page 171*	..			1,000	(
	,, SALARIES, WAGES AND RETIRING ALLOWANCES—					
	Salaries	2,116	18	0		
	Retiring Allowances	1,605	0	0		
	Clerks and Messengers	5,838	15	2		
9,319					9,560	13
	,, PREMIUMS ON POLICIES FOR STAFF PENSIONS—					
1,260	Portion paid by The Institution	..			1,058	7
	,, STATIONERY, POSTAGES, ETC.—					
	Stationery and Printing	1,231	19	2		
	Postages, Telegrams and Parcels	1,039	4	7		
2,329					2,271	3
	,, PUBLICATIONS ACCOUNT—					
12,000	Amount provided for the year, *per page 171*	..			12,000	0
	,, RESEARCH RESERVE—					
2,600	Amount provided for the year, *per page 177*	..			2,600	0
	,, LIBRARY—					
	Books and Periodicals	418	18	4		
	Binding	150	7	7		
	Clerical Pay, Pensions Premiums and Assistance	1,110	3	2		
1,499					1,679	9
	,, EXAMINATION EXPENSES—					
	Examiners, Printing and General	1,877	14	8		
	Salaries, Clerical Pay and Pensions Premiums	1,725	18	4		
	Postages	122	16	8		
3,723					3,726	9
1,412	,, CONVERSAZIONE AND ANNUAL DINNER	..			1,190	1
70	,, DIPLOMAS	..			58	4
	,, LOCAL ASSOCIATIONS—					
1,376	Grants to Local Associations, etc.	..			1,387	10
	,, CONTRIBUTIONS TOWARDS ADVISORY COMMITTEES IN					
105	THE DOMINIONS	..			60	0
	,, GRANTS AND CONTRIBUTIONS—					
	Engineering Public Relations Fund	101	3	6		
	British Standards Institution	50	0	0		
	Engineering Joint Council	12	10	0		
	Westminster Hospital	10	10	0		
	World Power Conference	21	0	0		
	Parliamentary Science Committee	10	10	0		
	Institution of Automobile Engineers Joint Meeting	5	0	0		
171					210	13
	,, LEGAL AND OTHER PROFESSIONAL CHARGES—					
	Legal Charges	134	1	10		
	Audit Fee	183	15	0		
	Engineers' Charges	5	5	0		
433					323	1
112	,, TRAVELLING EXPENSES TO COMMITTEES	..			124	7
—	,, OTHER TRAVELLING EXPENSES	..			212	10
—	,, GLASGOW CONGRESS	..			86	8
84	,, INTEREST ON LOANS	..			60	16
31	,, MEMORIAL SERVICES ADDRESSES, ETC.	..			—	—
215	,, CORONATION EXPENSES	..			—	—
					45,787	6
—	,, BALANCE, BEING EXCESS OF INCOME OVER EXPENDITURE FOR THE YEAR AS PER BALANCE SHEET, page *169*	..			1,330	2
£45,718					£47,117	8

OM 1st APRIL, 1938, TO 31st MARCH, 1939.

INCOME.

	£ s. d.	£ s. d.	1937-38 £
Subscriptions received applicable to the Financial Year 1938-1939	..	34,945 14 9	*33,713*
Entrance Fees	.	6,426 10 0	*5,006*
Life Composition	.	200 0 0	—
Interest, Dividends, etc.—			
On Institution Investments	1,200 18 3		
On Current Account	7 12 10		
Income Tax refunded for the year 1937–8	345 17 11	1,554 9 0	*1,692*
Examination Fees for the October, 1938, and April, 1939, Examinations	..	3,963 10 0	*3,577*
Library Fund Donations	..	27 4 6	*68*
Balance, being Excess of Expenditure over Income for the Year	..	— — —	*1,662*
		£47,117 8 3	*45,718*

TRUST[illegible]

CAPITAL ACCOUNTS AND INVESTMENT THEREOF AND INVESTMENT [illegible]

Capital Accounts. £ s. d.		Investments. Capital. £ s. d.	Unexpended Income. £ s. d.
8,038 9 4	TELFORD FUND.		
	£8,738 13s. 0d. 2½% Consols . .	7,988 9 4	
	£50 16s. 11d. 3½% War Loan . .	50 0 0	
270 0 0	MANBY DONATION.		
	£250 London & North-Eastern Railway 4% 2nd Guaranteed Stock	270 0 0	
6,337 12 4	MILLER FUND.		
	£5,129 17s. 5d. 2½% Consols . .	4,850 2 4	
	£1,513 15s. 9d. 3½% War Loan. .	1,487 10 0	
500 0 0	HOWARD BEQUEST.		
	£352 11s. 5d. 2½% Consols . . } Cost of Medal Die . . . . }	500 0 0	
600 0 0	TREVITHICK MEMORIAL.		
	£103 2½% Consols	100 0 0	
	£506 5s. 7d. 3½% Conversion Loan 1961.	500 0 0	
540 0 0	CRAMPTON BEQUEST.		
	£512 15s. 11d. 2½% Consols. . .	500 0 0	
	£40 13s. 7d. 3½% War Loan . .	40 0 0	
1,234 14 0	JAMES FORREST LECTURE AND MEDAL FUND.		
	£465 Southern Railway 4% Debenture Stock	604 14 0	
	£667 5s. 8d. 3½% War Loan . .	630 0 0	
1,647 10 10	PALMER SCHOLARSHIP.		
	£1,650 10s. 0d. 3% Redemption Stock, 1986–1996	1,547 10 10	
	£100 9s. 8d. 3½% War Loan . .	100 0 0	
1,080 0 0	JOHN BAYLISS BEQUEST.		
	£1,013 17s. 10d. London County 3% Stock, 1920	1,000 0 0	
	£80 7s. 10d. 3½% War Loan . .	80 0 0	
1,318 11 8	THE INDIAN FUND.		
	£1,353 4s. 2d. 2½% Consols . . .	1,148 11 8	
	£171 13s. 3d. 3½% War Loan . .	170 0 0	
1,000 0 0	VERNON-HARCOURT BEQUEST.		
	£1,082 9s. 10d. London County 3% Stock, 1920	1,000 0 0	
22,566 18 2	*Carried forward*	22,566 18 2	

FUNDS.

OF UNEXPENDED INCOME AT 31ST MARCH, 1939.

Capital Accounts.				Investments. Capital.			Unexpended Income.		
£	s.	d.		£	s.	d.	£	s.	d.
22,566	18	2	*Brought forward*	22,566	18	2			
1,300	0	0	WEBB BEQUEST.						
			£1,055 7s. 2d. Metropolitan Water Board 3% "B" Stock	1,000	0	0			
			£303 16s. 2d. 3½% War Loan	300	0	0			
2,733	1	10	WILLIAM LINDLEY FUND.						
			£1,214 London Midland & Scottish Railway 4% Debenture Stock	1,584	16	8			
			£1,109 2s. 9d. 4½% India Stock, 1950–1955	1,005	15	2			
			£151 9s. 7d. 3½% War Loan	142	10	0			
725	0	0	KELVIN MEDAL FUND.						
			£757 18s. 11d. 3½% War Loan	725	0	0			
4,250	0	0	CHARLES HAWKSLEY BEQUEST.						
			£955 Metropolitan Water Board 3% "B" Stock	573	0	0			
			£500 South Essex Waterworks 5% Preference Stock	435	0	0			
			£70 5% Sheffield Corporation Water Annuities; £40 4% Sheffield Corporation Water Annuities	1,992	0	0			
			£1,257 17s. 4d. 3½% War Loan	1,250	0	0			
1,101	6	5	COOPERS HILL WAR MEMORIAL.						
			£1,239 15s. 0d. 3½% War Loan	1,101	6	5			
			£215 12s. 3d. 3½% War Loan	..			214	8	2
3,570	4	0	C. C. LINDSAY CIVIL ENGINEERING SCHOLARSHIP FUND.						
			£3,500 3½% War Loan	3,570	4	0			
320	0	0	BAKER MEDAL FUND.						
			£290 5s. 8d. London County Cons. 4½% Stock, 1945–85	320	0	0			
809	11	0	JAMES ALFRED EWING MEDAL FUND.						
			£798 12s. 9d. Middlesex County Council 3% Stock, 1961–1966	807	0	0			
641	4	10	G. H. DENNISON FUND.						
			£706 3s. 2d. 3% Local Loans	641	4	10			
38,017	6	3	*As per Balance Sheet, page 169*	*38,015	4	3	*214	8	2

NOTE.—**The value of these Investments at ruling prices on 31st March, 1939, amounted approximately to £33,311 and £205 respectively.*

TRUST FUNDS INCOME ACCOUNTS FROM

Trust Fund.	Balance at 1st April, 1938. £	s.	d.
Telford Fund	99	10	10
Manby Fund	8	14	2
Miller Fund	382	4	11
Howard Bequest	0	8	2
Trevithick Memorial	5	7	5
Crampton Bequest	7	13	0
James Forrest Lecture and Medal Fund	15	2	7
Palmer Scholarship Fund	105	9	9
John Bayliss Bequest	52	19	6
Indian Fund	38	16	11
Vernon-Harcourt Bequest	123	6	9
Webb Bequest	212	10	9
William Lindley Fund	690	3	3
Kelvin Medal Fund	124	17	0
Charles Hawksley Bequest	124	0	1
Coopers Hill War Memorial Fund	333	19	11
C. C. Lindsay Civil Engineering Scholarship Fund	372	7	1
Baker Medal Fund	9	3	2
James Alfred Ewing Medal Fund	33	8	0
G. H. Dennison Fund	1	0	1
R. E. S. Cooper Legacy	25	0	0
Totals	£2,766	3	4

COMMITTEE ON THE DETERIORATION OF

ACCOUNT FROM 1ST APRIL, 1938,

	£	s.	d.
To Amount paid on behalf of or to the Committee during the year to 31st March, 1939	691	8	10
,, Balance *carried down*	1,391	7	1
	£2,082	15	11

RESEARCH

	£	s.	d.	£	s.	d.
To RESEARCH—						
Vibrated Concrete Research	20	0	0			
Pile Driving Research	250	0	0			
Earth Pressures Research	152	15	0			
Wave Pressures Research	249	11	3			
Simply Supported Steel Bridges Research	35	0	0			
Fish Passes Research	74	9	9			
Soil Corrosion Research	375	10	0			
Reinforced Concrete Structures Research	18	4	4			
Repeated Stresses Research	51	7	6			
Breathing Apparatus Research	22	6	6			
Earthing to Water Mains Research	300	0	0			
Special Cements Research	26	3	0			
Welding Research	250	0	0			
Velocity Formulas Research	71	17	8			
Joint Committee on Materials and their Testing	10	0	0			
To OTHER EXPENSES—				1,907	5	0
Travelling Expenses of Committee	204	4	8			
Salaries, Clerical Pay and Pensions Premiums	745	8	5			
Research Report	86	18	8			
Sundries, including Printing, Travelling, etc.	119	7	0			
				1,155	18	9
				3,063	3	9
To BALANCE carried down				2,962	10	3
				£6,025	14	0

1ST APRIL, 1938, TO 31ST MARCH, 1939.

Income: Including Income Tax refunded for the year 1937-1938. £ s. d.	Expenditure on Scholarships, Prizes, Lectures, etc. £ s. d.	Balance at 31st March, 1939. £ s. d.
220 14 7	284 6 9	35 18 8
9 15 11	7 4 1	11 6 0
183 1 4	112 5 3	453 1 0
8 16 5	— — —	9 4 7
20 2 1	11 17 0	13 12 6
14 5 8	20 18 2	1 0 6
41 11 4	47 9 6	9 4 5
28 3 2	63 15 0	69 17 11
32 14 10	30 0 0	55 14 4
40 0 4	10 6 6	68 10 9
32 4 10	63 0 0	92 11 7
42 10 5	— — —	255 1 2
104 12 6	40 0 0	754 15 9
27 2 6	41 17 6	110 2 0
204 3 7	182 14 6	145 9 2
51 10 2	46 7 10	339 2 3
124 5 0	25 0 0	471 12 1
12 18 10	— — —	22 2 0
23 10 4	24 15 0	32 3 4
19 5 9	11 0 0	9 5 10
— — —	25 0 0	— — —
£1,241 9 7	£1,047 17 1	†£2,959 15 10

† Of which £214 8*s.* 2*d.* is invested (*see page 175*).

STRUCTURES EXPOSED TO SEA ACTION.

TO 31ST MARCH, 1939.

	£ s. d.
By Balance, as per last Account	2,047 14 5
,, Subscriptions	25 1 6
,, Interest on Deposit	10 0 0
	£2,082 15 11
,, Balance *brought down as per Balance Sheet, page 168*	1,391 7 1

RESERVE.

	£ s. d.
By Balance, as per last Account	1,865 4 0
,, Contributions from other Bodies	1,560 10 0
,, Institution Revenue Account—Amount provided for the year—*per page 6*	2,600 0 0
	£6,025 14 0
,, Balance *brought down as per Balance Sheet, page 168*	£2,962 10 3

INSTITUTION INVESTMENTS AT 31st MARCH, 1939 (INCLUDING THOSE HELD IN RESPECT OF REPAIRS AND RENEWALS RESERVE) AT COST.

£	s.	d.		£	s.	d.
3,000	0	0	Metropolitan Water Board 3% "B" Stock	2,958	16	0
6,000	0	0	London and North Eastern Railway 4% Debenture Stock	7,749	18	3
6,000	0	0	London Midland and Scottish Railway 4% Debenture Stock	7,452	14	8
2,545	0	0	London Midland and Scottish Railway 4% Guaranteed Stock	1,976	7	10
5,994	15	2	3½% War Loan	3,586	11	4
2,720	5	5	London Passenger Transport Board 4½% "A" Stock .	3,327	9	3
3,809	0	2	3½% War Loan	3,824	8	6
452	0	0	London Midland and Scottish Railway 4% Guaranteed Stock	351	3	7
989	14	7	London Passenger Transport Board 4½% "A" Stock .	1,210	12	11
5,327	6	5	New Zealand 3% Stock, 1952–1955	5,334	7	6
9,400	0	0	Middlesex County Council 3% Redeemable Stock, 1961–1966.	9,391	11	6
470	4	3	3% Local Loans	461	3	0
			National Gas and Oil Engine Co., Ltd., 3,336 Ordinary Shares of £1	2,668	16	0
			As per Balance Sheet, page 169 . .	£50,294	0	4

NOTE.—*The value of these Investments at ruling prices on 31st March, 1939, amounted approximately to £42,998.*

JOINT MEETING WITH THE BRITISH SECTION, SOCIÉTÉ DES INGÉNIEURS CIVILS DE FRANCE.

10 May, 1939.

WILLIAM THOMSON HALCROW, President, British Section, Société des Ingénieurs Civils de France, in the Chair, supported by

WILLIAM JAMES EAMES BINNIE, M.A., President Inst. C.E.

"Photo-Elasticimetry: its Application to the Measurement of Deformations of Bridges."

By Monsieur ROBERT FLAMENT-HENNEBIQUE.

(*Abridged* [1].)

THE bridge at Saint-Thibault-sur-Loire possesses two features of special interest, in that it was the first bridge entirely built in pervibrated concrete, and, in addition to the usual tests, it was subjected to a thorough auscultation by means of a new process, namely, photo-elasticimetry. It is one of the four structures spanning the Loire between the departments of Cher and Nievre. The new structure replaces a suspension bridge, over one century old, the piers of which were utilized for the new work. The selected design was a ferro-concrete bridge as light as possible in appearance, consisting of a series of arches carrying an upper deck by means of vertical stanchions.

Local circumstances led to the adoption of a rise of 19 feet 9 inches for the arches, which have spans of 209 feet 3 inches and 215 feet 9 inches respectively. The total length of the deck is 1,327 feet 8 inches between abutments. The decking comprises a roadway 19 feet wide, carried on transverse secondary beams and longitudinal main beams, the latter being supported on the uprights which transmit the load on to the arches. The footpaths, 3 feet 8 inches wide, have ferro-concrete handrailing and are carried on cantilevers.

The old foundations were retained, but both the abutments and the piers were strengthened by driving a ring of concrete piles connected to a ferro-concrete waling. The space between each old foundation and its

[1] Copies of the Paper and Discussion may be obtained on loan from the Loan Library of the The Institution; a limited number of copies is also available, for retention by members, on application to the Secretary.

new ring of piles was filled up with mass concrete. Stone pitching is laid around the ring to prevent erosion, and the penetration of the piles in the sandy strata is such as to prevent any spewing of the sand under the masonry foundations, and therefore to guard against settlement.

In view of the rapid rise in the level of the water at flood-times and of the magnitude of such rising, it was decided at the outset to dismiss any thought of resorting to semi-articulation of the arches, but to seek the minimum angular variation of the section at the bearing by studying the flexibility of the arches, and varying the moment of inertia. The depth of the arches decreases gradually towards the bearings, thereby gradually reducing the vertical rigidity, and securing an easy deformation without generating important reactions of fixation. On the other hand, the rise of the arches was reduced by increasing the moments of inertia in proportion to the bending moments, the maxima occurring at the haunches.

The depth of the rib, which is the determining factor of the moment of inertia, varies from a maximum of about 4 feet at the haunches to a minimum of 16 inches at the springing. The normal thrust being a maximum at the support, the average compressive stress is maintained constant by gradually splaying out the ribs at the springings. This arrangement provides, in addition, adequate transversal rigidity to the three arches and guards against any deformation under the pressure arising from the flow of the river. This transverse rigidity is completed by the provision of bracings which contribute to ensuring indeformability.

After reference to the bearing on the piers, the decking, and the flood opening, the construction of the temporary bridge was described. It was out of the question to stop the heavy traffic which was using the old suspension bridge during the construction of the new bridge.

The execution of the work requires no special comment. The materials required for the new bridge were 400 tons of steel and 3,144 cubic yards of concrete, aluminous cement being used and special steps being taken to overcome the rise in temperature by watering. In addition to increasing the compressive working stress by the use of aluminous cement, the method of internal vibration, or pervibration, was adopted. The tests of concrete during the construction were carried out on 8-inch cubes made with concrete taken from the mass actually worked in position, and indicated a crushing strength ranging from 7,100 to 8,800 lb. per square inch; it was therefore possible to adopt a working stress of 1,850 lb. per square inch.

Tests after 1 week and after 90 days yielded results as set out in Table I.

Pervibration was effected by means of compressed-air vibrators. A compressed-air vibrator comprises an eccentric mass revolving rapidly inside an envelope of suitable shape, which is thereby put into a state of intense vibration. When the vibrating tool is introduced into a mass of concrete it induces a vibratory state, the effect of which extends across a

TABLE I.

Test.	Time: days.	Material test.	Crushing strength: lb. per square inch.
1	7	Pervibrated concrete, not watered during settling	3,260
2	7	Not pervibrated, but watered	6,025
3	7	Pervibrated and watered	6,950–8,080
4	90	Not pervibrated, but watered	7,100
5	90	Pervibrated and watered	8,300–8,800

zone of influence of dimensions varying in accordance with the shape and dimensions of the envelope, the type of vibrator, and the proportions of the concrete. Within that zone, the aggregate elements of the concrete are very closely compacted and the mortar surges up, even when the mix is very dry and comparatively poor in cement. The pervibrated mass moreover acquires a temporary state of liquefaction, so that buoyancy enters into play as regards the pervibrator, which, if its weight and dimensions have been properly adjusted, rises automatically when further concrete is added to fill up the mould. The new concrete also rapidly acquires the liquefied state.

Experiments proved that, with the use of pervibration, and without taking any special precautions, it is easy to obtain a degree of compactness of 85 per cent. for the concrete, whereas it is very difficult to obtain more than 75 per cent. in actual practice with hand-ramming, even if granulometry has been carefully studied. Consequently the voids are reduced to the extent of 40 per cent. by pervibration. This allows a higher working stress to be adopted.

It may be asserted that, in every case, pervibration gives better results than hand-tamping, and the improvement is quite apparent even in the case of the test cubes, to which greater care can be given, when hand-tamping, than is possible when actually ramming concrete in situ. It must be admitted, however, that, if pervibration is an important factor of improvement, the progress is limited to the utilization of concrete; in other words, it counteracts only the deficiencies in the most important constituent, namely, the cement.

For a long time research has been conducted toward the production of a cement devoid, or almost devoid, of shrinkage, but until the desired material is finally produced, shrinkage must be counteracted by some empirical treatment, which consists generally in avoiding too rapid a desiccation during the hardening period. This, however, is merely a palliative, and, despairing of ever obtaining a cement devoid of shrinkage, certain engineers considered the possibility of reversing the position, and of using a cement having variations of volume which, as time goes on, would no more be a nuisance, but, on the contrary, an advantage. This led to the idea of expanding cement.

Mr. Hendricks is at present experimenting in France on the production of such a cement. His principle is to impede the normal shrinkage, due to the desiccation of the capillary cavities, by retarding the dissolution of certain anhydrous constituents of the cement, and by inducing the formation of crystals which fill up the voids left by the disappearance of the water. Such a cement would have many advantages in fixed and bowstring arches, in reservoirs, and in underground work such as tunnels, foundations, wells, etc., and also where new concrete extensions are built on to existing structures. For some time to come this kind of cement may still have to be considered as a laboratory product, but, pending its general use, processes such as pervibration are well worth utilizing.

Photo-elasticimetry is a process of direct testing allowing the direction and magnitude of the stresses which develop in any part of a structure to be ascertained. After referring to the properties of polarized light, the Mabboux photo-elasticimeter and its application to the St. Thibault bridge was described. The method consisted of fixing small silver mirrors on one of the faces of every one of the three arches in the span, in the sections for which calculations were made, and as near as possible at the level of the upper and of the lower reinforcement. In order to eliminate, as far as possible, the disturbing shape of the constituent elements and of the grain of the mortar, it was advisable to make the dimensions of the mirrors negligible in comparison with those of the corresponding member, but very great in comparison with those of the grain of the mortar. After many preliminary experiments, it was decided to provide mirrors about $\frac{19}{32}$ inch in diameter and about $\frac{1}{8}$ inch thick. These mirrors were sealed on the face of small aluminous-cement concrete blocks of the same mixture as that of the corresponding members, these blocks being concreted in, to the number of fifty-two per arch. When the shuttering was removed these mirrors were flush with the external face of the ribs.

Tests were carried out and the working stresses in the various sections were calculated from the actual readings. It was at once obvious that the working stresses revealed by photo-elasticimetry were much lower than those indicated by the calculations. This was not surprising, and there were two explanations of the fact. First, calculations took into account the influence of shrinking and the temperature-stresses. During the tests, the differences in temperature were negligible, and as the main shrinkage occurred during the hardening of the concrete, its effect during the tests is also negligible.

On the other hand, calculations take into account particularly unfavourable conditions which are extremely unlikely to occur simultaneously when the tests are carried out. If it is possible, in a laboratory, to place a test piece under conditions identical with those assumed for calculation purposes, it is impossible to do the same when dealing with an important

structure, and especially when there is a probability that the unfavourable conditions may never be all realized at the same time.

Table II shows typical results obtained in various sections by elasticimetry, and those obtained by calculation.

TABLE II.

Central rib.			External rib.		
Number of section.	Static loading.		Number of section.	Static loading.	
	Calculations: kilogrammes per square centimetre.	Photo-elasticimeter: kilogrammes per square centimetre.		Calculations: kilogrammes per square centimetre.	Photo-elasticimeter. kilogrammes per square centimetre.
2	128	46	2	108	70
3	130	20	4	109	56
5	128	62	6	116	58

All these results, excepting that corresponding to section No. 3 of the central rib, compare fairly well together, the working stress from calculations being, on an average, about twice as great as that revealed by photo-elasticimetry.

It must be understood, however, that this was the first large-scale practical application of the new process, which has been applied previously only in the laboratory research. It is likely that the further laboratory work at present being undertaken may lead to improvements in the apparatus which will give to the process its full value as a method of control.

In any case, the results so far obtained are in full accord with the logical considerations based upon the laws of elasticity, both as regards the direction and the magnitude of the stresses. They give, in addition, valuable information regarding the stresses observed in normal service, and, consequently, regarding the factor of safety of the work. Further, as the mirrors are fixed permanently in the structure, they will facilitate a new auscultation of the structure as a whole, or of any of its elements, should this become necessary in the course of time.

In conclusion, it was pointed out that a lapse of time will be necessary to enable judgement to be passed on the value of the results obtained, but the possibilities offered for the auscultation of stresses in actual structures, either by acoustic methods or by those of photo-elasticimetry, will gradually enable the test-results to get nearer and nearer to the truth.

A vote of thanks to the Author for his Paper was proposed by Mr. W. T. Halcrow, President, British Section, Société des Ingénieurs Civils de France, and was seconded by Mr. W. J. E. Binnie, President Inst. C.E.

In the ensuing Discussion seven speakers took part. It was suggested in connexion with the stress-measurements that the material in the bridge was stressed simultaneously in several directions, whereas the test-piece in the measuring apparatus was only stressed in one direction, and that stresses actually measured were those due to live loading on the bridge. Different observers might vary in their perception of colour. It was also pointed out that what was actually measured was the difference in the principal stresses at the point concerned, and that the value of the measurements would be enhanced if the mirrors in the material being investigated could be rotated in orientation with the principal stresses. The strength of the concrete, and the allowable working stress, were also discussed.

Paper No. 5152.

"The Principles of Drag-Suction Dredging."

By HERBERT CHATLEY, D.Sc. (Eng.), M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

INTRODUCTION: THE DRAG-SUCTION DREDGER *CHIEN-SHE*.

In the Author's Paper "Energy Considerations in Dredging"[2] reference was made to the mechanical conditions of drag-suction dredging, based on a study of the available data extending over 10 years. Since writing that Paper the largest drag-suction dredger in operation (the *Chien-She*, working in the Yangtse estuary) has been built by Messrs. Schichau of Danzig and Elbing for the Whangpoo Conservancy Board to the Author's specification (Fig. 1, Plate 1) and has been working successfully on the entrance bar of the Yangtse river since the summer of 1935. The annual output is over 5,000,000 cubic yards of in-situ material. Just completed is a second and rather larger dredger (the *Fu Shing*), which is provided with additional propulsive power, water-tube boilers, and cylindrical dumping valves.[3]

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[1] Correspondence on this Paper can be accepted until the 15th August, 1939, and will be published in the Institution Journal for October 1939.—SEC. INST. C.E.

[2] Inst. C.E. Selected Engineering Paper No. 125, 1932.

[3] For description, see *Engineering*, vol. CXLVI (1938), pp. 495 (28th October), 577 (18th November), and 638 (21st December).

The design and operation of the dredger *Chien-She* involved the investigation of various matters concerning which little information was available, and it is hoped that some of the results may be of general interest and utility.

The original Frühling patent [1] indicates the basic principle of combined scooping and suction, but the inventor, rather naturally, does not seem to have appreciated all the mechanical problems involved. The success of his dredgers from 1902 onwards in handling very soft mud led to the type being widely used for work on ocean bars, and there are now about seventy such vessels in existence. It was found, however, that the output was disappointing whenever sandy material was met with, or when the mud to be excavated was very stiff. Herr Otto Frühling and his imitators made various efforts to overcome this difficulty, but to this day the thickness of the pumped mixture obtained by this type of dredger in sand is often not very much better than that obtainable with the ordinary moored suction-dredger. In the case of fine sand which does not settle quickly, a thin mixture is particularly objectionable in hopper-dredgers since it involves the useless transport in the hopper of a large volume of water or, alternatively, the flowing overboard of the thinner spoil in the hope of retaining a fair load of coarser particles. Much of the materials sent overboard actually resettles in the cut, even when there is a strong transverse current. On the other hand, with this type of dredger working in smooth, soft mud, it can happen that the material comes up almost free of added water.

The case of the Yangtse bar mud was peculiar inasmuch as the typical bar-material was smooth and soft but of unusually high density (1·6 to 1·9), in spite of the mineral being of the usual silica and alumino-silicate make-up. In actual fact this material proved extraordinarily amenable to drag-suction, being brought to the surface with a density at times as high as 1·6. Whole hopper loads with a density of this value have been obtained; those with 1·5 are frequent and average hopper densities of less than 1·4 are considered bad. Assuming the in-situ material density to be 1·8, a density of 1·6 corresponds to a mixture in the hopper containing 75 per cent. of in-situ material [2], 1·5 corresponds to 62·5 per cent., and 1·4 corresponds to 50 per cent., proportions quite unobtainable with moored suction-dredgers.

In the middle of the bar, however, a patch of sand with a density of about 2·0 was found, and it was at first quite difficult to dredge mixtures with a density exceeding 1·15, corresponding to 15 per cent. of in-situ material. Fortunately this sand area was limited and its margin consisted of loam which pumped more easily. It was later found that by partially closing the mouth of the draghead a richer mixture up to a density of approximately

[1] Brit. Pat. No. 19239, Sept. 9, 1898.

[2] Proportion of original material of density s_1, in a mixture of density s_2, $= \dfrac{1 - s_2}{1 - s_1}$.

1·25 could be obtained, but the additional resistance to cutting remained a difficulty [1]. Intermediate materials gave better results.

Relation of Output to Draghead Dimensions and Vessel Speed.

With one of the dragheads in use there is an entrance area of about 20 square feet and the speed over the bottom during dredging may be as much as 3 knots (say 5 feet per second). The ideal output in this case would be 100 cubic feet per second if the head were completely buried in the mud. The dredger being designed to take 2,500 cubic yards per trip, this would mean that the hopper might be filled in

$$(27 \times 2{,}500) \div 100 = 675 \text{ seconds or } 11{\cdot}25 \text{ minutes.}$$

Such a good result as this was not actually obtained, but occasionally hoppers were actually filled with 4,000 tons of mixture in 15 minutes, consisting of about 2,500 cubic yards (3,375 tons) of in-situ mud and 625 tons of water (833 cubic yards), that is 33 per cent. by volume of water added, corresponding to a mixture density of 1·6 if the in-situ density were 1·8, or 1·525 if the in-situ density were 1·7.

If all the added water is obtained by suction (pressure water could be supplied but was not always found necessary), this means that in the average, the draghead was 25 per cent. (11·25 minutes divided by 15 minutes is 75 per cent.) open at the top and the average inflow-speed of the water was about the same as that of the mud, since the mixture consisted of about 25 cubic feet per second of water and 75 cubic feet per second of mud. Actually the water-inflow must fluctuate according to the effective suction at the mouth of the draghead and the breaking up of the mud in the lower part of the pipe.

This roughly represents the most favourable conditions.

Turning now to the case of sand, the higher resistance of the sand to cutting has two results. Firstly, the draghead does not penetrate so deeply into the bottom, leaving a far larger opening for the water to enter the head. Secondly, the speed of the ship over the bottom will be reduced. Thus if the buried sectional area is reduced to, say, 8 square feet and the speed is reduced to 1·5 knot (2·5 feet per second), the output cannot exceed 20 cubic feet per second. The inflow-speed of the water will be higher than that of the sand owing to the large opening (12 square feet) and the strong vacuum, and may easily amount to 10 feet per second, making an influx of water of 120 cubic feet per second and causing the mixture to have a

[1] Engineers in the United States of America use a flat shoe-shaped head with a grid that scrapes up the sand. This head lies behind the pipe, thus avoiding the bend but losing the plough effect.

density of 1·14. This is precisely the sort of result that is obtained in sand if the same head is used as for mud.

It is obvious that if more thrust is available the head can be buried deeper, but, unless the dredger is designed with a surplus of thrust, this is not feasible. In actual fact with a given dredger there is only one method of dealing with the situation, and that is to use a smaller draghead or partially to close the opening with adjustable shutters (*Fig. 2*). Both devices are quite successful and can increase the density of the sand mixture to 1·25 or even 1·3. The actual rate of dredging is not thereby perceptibly increased, but the denser mixture gives a better hopper-load, involving less wasteful transport of dilution water and so increasing the total daily output. The Author believes that with optimum thrust and area combination a sand mixture containing nearly 50 per cent. of original material (by volume) could be obtained, since mixtures of this kind can be pumped under pressure.

Resistance to Cutting.

It is clearly necessary that there shall be a reserve of propeller-thrust, over and above that required to move the ship through the water at the relative speed (in the case of strong tidal currents this means full current-speed plus speed over the bottom), sufficient to force the draghead into and through the bottom. Actual towing tests are the best means of measuring the thrust required, but an approximate idea can be gained by multiplying the buried periphery of the draghead by 3 times the buried depth into a shearing force appropriate to the material. Thus with a head 8 feet wide buried 2 feet deep in mud having a shearing resistance of 300 pounds per square foot, the total resistance is about

$$(8 + 2 + 2)(3 \times 2) \times 300 = 21{,}600 \text{ lb., or say 10 tons.}$$

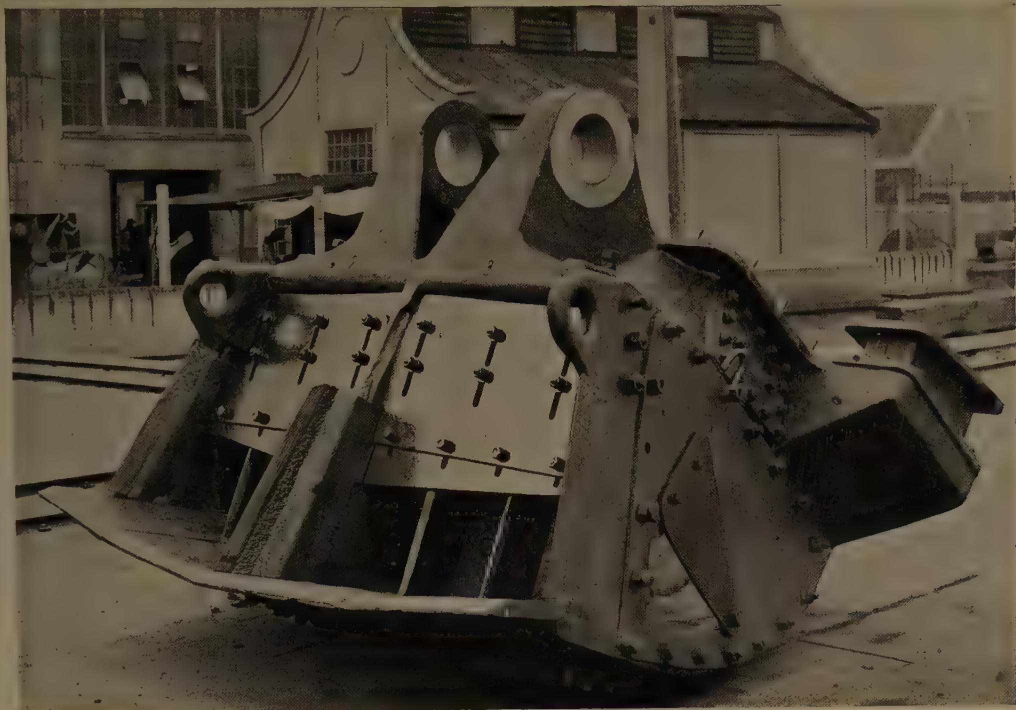
With a speed over the bottom of 3 knots (5 feet per second) this amounts to

$$\frac{21{,}600 \times 5}{550} = \text{nearly 200 horse-power.}$$

In sand, if the same head is only buried 1 foot and the shearing resistance is 700 lb. per square foot, the total resistance is

$$(8 + 1 + 1)(3 \times 1) \times 700 = 21{,}000 \text{ lb.}$$

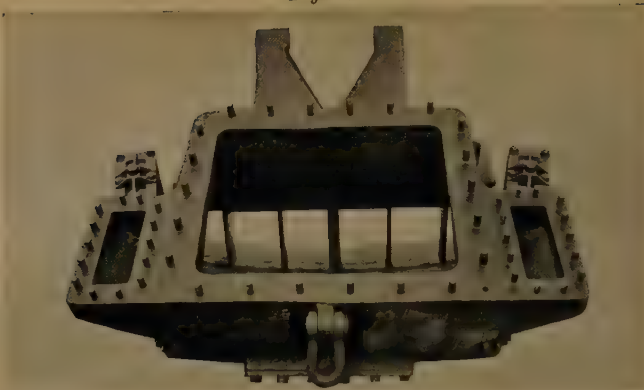
This example indicates very clearly how greater resistance reduces the penetration and, in the absence of shutters, increases the water opening.



STANDARD DRAGHEAD WITH SHUTTERS.

5152.

Figs. 3.



Top view.

Threequarter top view.

Front view.

EXPERIMENTAL DRAGHEAD.

RESISTANCES IN SUCTION PIPE.

The vacuum at the eye of the pump has to overcome :

(*a*) The head of fluid in the suction pipe less the hydrostatic head of water.

This may be quite important with thick mixtures. Thus if the eye of the pump is just at water-level (it is usually below that level, but as the impeller is vertical and nearly 9 feet in diameter it is difficult to put the eye very much below the light-draught water-level) and the mixture has a density of 1·6, the head to be overcome measured in feet of water is 0·6 times the depth in feet of the bottom of the cut below water-level. For the vessel in question this depth was a maximum of 45 feet. Since $0{\cdot}6 \times 45 = 27$ feet (24·5 inches of mercury) this would utilize almost all the vacuum, and indicates clearly that this type of dredger cannot work to great depth if it is to lift dense mixtures. In actual fact, when the hopper was full of water, the eye of the pump was some 11 feet below water-level, so that the lifting head required in this state was :

$$\{0{\cdot}6 \times (45 - 11)\} - 11 = 9{\cdot}4 \text{ feet.}$$

At light load the eye of the pump was actually 3 feet below water-level so that the maximum lifting head required to overcome the weight of the mud in the pipe was

$$\{(0{\cdot}6 \times (45 - 3)\} - 3 = 22{\cdot}2 \text{ feet.}$$

Obviously this great demand on the vacuum tends to make the mixture thinner at the beginning of the pumping.

(*b*) The friction head in the pipe.

For a thick but smooth-running mixture the fluid friction is about 10 times that of clean water, but the pipe has one sharp bend in it, at the drag-head, and certain changes of section, so that about 15 times is the least that should be considered, or say 0·075 lb. per square foot at a speed of 1 foot per second. The actual length of the suction pipe in question was about 75 feet and the velocity (for mud mixtures) was of the order of 10 feet per second. The periphery was about 11 feet, so that the total resistance was about

$$75 \times 11 \times 0{\cdot}075 \times 10 \times 10 = 6{,}188 \text{ lb.}$$

The sectional area of the pipe being about $10\frac{1}{4}$ square feet, this corresponds to about 600 lb. per square foot of pipe section or 4 lb. per square inch, equalling about 25 per cent. of an atmosphere or 8 feet of water (or 7·5 inches of mercury).

If the mixture is very thick or the particles inclined to lock together the coefficient of rubbing friction will increase and the flow may be checked by the lack of suction head.

(*c*) The kinetic head.

The main pump of the *Chien-She* when taking in water alone brings up 18,000 tons per hour, or 5 tons per second, equalling 180 cubic feet per second. This figure may be compared with the dense mixture referred to above (2,500 cubic yards of mud plus 833 cubic yards of water in 15 minutes equals 100 cubic feet per second). The water intake velocity for 1 inch (mercury) of vacuum at the draghead is, allowing a coefficient of contraction of say 0·6, about 9 feet per second, and rises as the square root of the available vacuum. Thus, even allowing for the higher densities, 2 inches of vacuum (say 2·1 feet of water) will provide the necessary kinetic energy.

If the mixture is thin and the mouth of the draghead is only partly buried, the vacuum in this place may rise to approximately 4 inches of mercury, causing an intake velocity of perhaps 20 feet per second, so accounting for the excess of water in sandy mixtures.

The dredger is provided with a high-pressure pump capable of giving as much as 6,000 gallons per minute at 30 feet head, or 2,500 gallons at 160 feet head. The first figure corresponds to about 16 cubic feet per second, which is quite a large fraction of the small dilution necessary with high-density mixtures. This water is fed into the back of the draghead by nozzles flush with the interior surface and directed as nearly as possible in the direction of the flow of the mud; that is, backwards into the head and pipe.

Design of Suction Pipe.

Having regard to the considerations of vacuum efficiency, the Author has been very surprised to note in many designs the lack of attention which is given to designing the head and suction pipe to have minimum fluid resistance. He is convinced that the pipe should be as large, as short, and as straight as possible. Single pipes are to be preferred to double ones (which have 41 per cent. more skin area for the same sectional area), the pipe should run direct into the eye of the pump, and the knuckle-joint which is required to allow the pipe to be lowered should be arranged so that the bend is as easy as possible in the average working position. At the junction with the draghead, the curve, which (including the back of the head) is necessarily nearly 180 degrees, should be of as large a radius as practical and the inside should be streamlined. It is doubtless preferable that the pipe should be circular in section rather than rectangular, but the great diameter (plus the width of the ladder frame which supports the pipe) affects the width of the well and indirectly the beam of the ship, so that a rectangular section with rounded internal corners is not so very objectionable. Nevertheless it is a fact that the perimeter for the same cross-sectional area is about 12 per cent. greater for a square section than for a circular one (the ratio is still larger for rectangular sections), and the

friction is increased in this ratio; there is further added hydraulic resistance at the fairing into the pump-eye, so that a circular pipe is definitely preferable. No pockets, lap-joints, or projecting bolt- or rivet-heads should interrupt the smoothness of the suction pipe. In particular, the inside of the draghead should be streamlined (not merely "faired off", but adapted to fluid flow without reverse eddies) so that there will be no check of any kind to the motion of the mixture. The early Frühling heads were rather defective in this respect, especially the very broad ones.

An external envelope for the pipe-ladder to reduce the fluid resistance to propulsion and currents is advantageous.

An interesting feature of this particular dredger is that the pressure-water pipe has been made a link which holds the head at a constant attitude by means of a fork, so that the cutting angle is not affected by the depth of the cut. The angle can be changed if desired by wedge-shaped packing pieces. Angular control-gear was considered but it was decided that its unknown advantages were outweighed by the loss of rigidity and the probable difficulty of maintaining a relatively delicate underwater device.

Figs. 3 (facing p. 189) show an experimental head which has been built, with removable upper plug-lip, suction-release spring-controlled side valves, and extra access-doors for possible future interior jets to be operated either by the suction or from the pressure-water line. The Author believes that in some instances such jets might be useful if externally streamlined and directed in the direction of the mud flow. Frühling contemplated various devices, such as rotary cutters inside the drag-bucket, but the Author is convinced that such apparatus, by obstructing the mud flow, would nullify any advantages it might otherwise possess, and that the head should be kept clear only by smoothness, stream-form, and jets parallel to and in the same sense as the flow.

Pumping Plant.

The main pumping engine is a triple-expansion steam engine of 2,400 indicated horse power at 150 revolutions per minute, the suction pipe is equivalent to a 43·275-inch diameter section, and the discharge pipes leading over the hoppers are each 27·625 inches in diameter. The hoppers can be pumped out by two suction pipes of same size as the discharge pipes. Overboard-discharges of 39·375 inches diameter are provided for emergency and shore pumping. The impeller of the main pump is four-bladed and is 8·5 feet in diameter over the tips. The water delivery is 18,000 tons per hour at 120 revolutions per minute. Details of this pumping plant are given in Figs. 4, Plate 1.

Full particulars of the vessel are contained in the Author's report

to the Conservancy Board [1] which has been published, slightly abridged in London [2].

DISCHARGING.

The vessel is ordinarily discharged through ten pairs of wooden doors the opening at each pair being 81 inches by 88 inches. In the case of mud the efflux is very easy and the whole hopper can be emptied in less than 2 minutes, but with sand the whole clearance may take 4 minutes. The vessel is provided with five sea intakes on each side controlled hydraulically from the bridge. These are intended to provide dilution water when pumping out the hopper, but in actual fact the mud can be discharged through them with comparative ease. In view of this fact it was decided to put cylindrical valves in the second dredger so as to reduce the cost and inconvenience of door maintenance and also to avoid the risk of striking an open door against the sea bed. The doors on the whole have proved satisfactory, but there is considerable wear at any point where a leak develops due to the erosive action of the sand on the timber. On one occasion the hydraulic ram controlling the door chains on one side failed to act, causing a dangerous list, and on another occasion the chain supporting a pair of doors broke above the balancing bridle and the bridle fell through.

When not dumping the main door chains are locked with heavy steel cotters, and the labour required to knock these cotters in and out is worth saving.

The objection to the cylindrical valves is that they occupy a considerable volume in the hoppers, and if the material is stiff it may jam between the sides of the hopper and the cylindrical wall of the valve. They have functioned quite well for many years working on the *Leviathan* in the Mersey when handling sand, and are reported from several ports to be quite satisfactory for use in mud dredging.

For discharge by pumping, hand-gear to control the exhausts to the suction pipe-lines have been provided and also hand-gear for the sprinkler controls, by which high-pressure water can be admitted to the hoppers just above the doors. This gear has worked satisfactorily, but actually the hoppers are rarely pumped out so that its trials have not been exhaustive. It would naturally be preferable that all these valves should be hydraulically controlled, and only the cost and complexity of the extra hydraulic pressure-lines and control gear on the bridge has caused hydraulic control to be omitted in this case. Labour in China is comparatively cheap and when

[1] The Dredger "Chien-She." Whangpoo Conservancy Board, Shanghai, 1936.

[2] "The Dock and Harbour Authority." Vol. 16 (1936–37), pp. 278 *et seq.*, and 308 *et seq.*

pumping out there is no such urgency as when dredging, so that the valves can be opened in succession.

Computation of Discharge.

It is of the highest importance that the actual output of the dredger should be rather accurately determined, and in any case not be overstated. Seeing that the hopper load always contains some added water (unless it is first pumped dry and the mud is sucked in with a completely buried head, and no pressure water is used), the problem is as follows:

Given the change of displacement during loading from the change of draught and the volume of mixture in the hopper from the level of its surface, to find the volume of in-situ material in the hopper-load.

The change of displacement shows the weight of the added load in tons, and the calibration of the hopper gives the volume of the mixture. If the latter is taken in cubic metres, the ratio of the added weight in tons to the hopper volume gives, with sufficient accuracy, the density. If the "in-situ" density is assumed (on the basis of previous observations), the fraction of the original material in the load can be simply computed or read off on a graph.

During the official contract trials actual measurements of the in-situ density were obtained by "dry" scrapings made with the draghead. These were dug out of the lifted head, placed in cylindrical measuring vessels and pressed into place by hand until the vessels were full and then weighed. As in actual fact such scrapings contained interstices and could not be brought back to the original consistency, these densities tended to be a little on the low side, but not much. The figure of 1·8 was often obtained for such mud, but occasionally lower figures occurred.

On the trials the actual densities so obtained were used in computation so that the contractors should not be penalized, but in practice the 1·8 reference density is usually employed, unless the material appears different.

The initial water in the closed hopper mixes with the load, but if the latter is dense the initial water tends to be pushed over the weirs when the hopper is nearly full and does not greatly dilute the mixture. It is, however, an additional disadvantage when pumping fine sand, but not usually sufficient to warrant the loss of time involved in pumping the hopper dry. Draught indicators are fitted, but are only used to control actual readings in the pipe well, fore and aft. When dealing with material which will pump well the load-level is regulated by a weir at a suitably low level, as a completely full hopper with such material gives too great a draught. With thinner mixtures the hopper can be filled to the coaming-level and run over it, but it should be emphasized that it is limiting displacement that should be aimed at and not simply full hoppers which may consist largely of water.

The displacement at 19 feet draught is about 9,000 tons (adjusted from time to time for consumption of coal and water), but would be only 7,400 tons if the doors were open.

The displacement at 11 feet draught is about 5,000 tons, but it is only 4,300 tons if the doors are open. In other words, the hopper contains 700 tons of water or 700 cubic metres if the hopper is not pumped out.

The hopper capacity to the deck-level (3 feet below the coamings) is 2,900 cubic metres, so that at the moment of filling, if the water has not been pumped out, there is only 2,200 cubic metres available. If the mixture has a density of 1·5 this means that 3,300 tons can be put in, but in actual fact the full 2,900 cubic metres of mixture can be pumped, adding $2,900 \times 1·5 = 4,350$ less the 700 tons of initial water pushed overboard, leaving an additional load of 3,650 tons.

If the density were 1·6, 3,540 tons could be put into the 2,200 cubic metres of space, or 4,640 tons if the water were pushed out, increasing the displacement by $4,640 - 700 = 3,960$ tons, or say 4,000 tons, which is the load for which the ship is designed.

A clearer idea of the effect of the hopper loading on the displacement may be obtained by considering the following routines :—

1. (*a*) Draught 11 feet. Door open. The displacement is 4,300 tons, equalling the weight of the ship.
   (*b*) Draught 11 feet. Doors closed. The weight of the ship is still 4,300 tons, but 700 tons of water is trapped in the bottom of the hoppers and the rated displacement is now 5,000 tons.
   (*c*) Water pumped out of hoppers. The ship rises until the draught is about 9·5 feet and the displacement is again 4,300 tons, but the whole space in the hoppers is now available to produce buoyancy.
   (*d*) 4,000 tons of mixture pumped in. The total displacement is now 8,300 tons and the draught is nearly 18 feet. The level of the fluid in the hopper depends on the density of the mixture.

2. (*a*) and (*b*) as in 1.
   (*c*) Hopper filled to coaming-level (3,240 cubic metres, less 700 cubic metres of water already in the hopper, equals 2,540 cubic metres of mixture). If the mixture poured in has a density of 1·5, the weight added is 3,810 tons and the total content of the hopper is 4,510 tons. The total displacement (doors necessarily closed) is 8,810 tons, the draught being about 18·5 feet. The mixture, including the original water in the hopper, has a mean density of 4,510 divided by 3,240, or 1·38.

3. (*a*) and (*b*) as in 1.
   (*c*) Hopper filled to coaming-level with pumped mixture, the original water being pushed overboard. If the density of inrun is

again 1·5, this means an addition of 4,860 tons to the light weight of the ship, so that the total displacement is 4,860 plus 4,300, which equals 9,160 tons, the draught then being about 19·2 feet.

Unless the hopper is pumped out there is always an ambiguity due to the lack of knowledge concerning the movement of the original water in the hopper. In addition to this the first part of the pump discharge is usually thin, and in actual fact the usual procedure is to run the pump until the hopper is full to the weir or coaming-level (according to the type of material being pumped) and to continue pumping until the draught reaches the safe maximum. (For this particular vessel owing to the shallowness of the dumping areas the draught was intended not to exceed 18 feet, but when tides permitted somewhat greater draughts were used.)

It is then assumed that the initial water and the starting flow have all been pushed overboard at the weir or coaming, so that the additional weight is the difference between the "light" draught displacement with doors closed (that is, the weight of the actual ship) and the loaded draught displacement. Provided that the pump is run until the draught is a maximum, the error in this assumption is not serious.

To illustrate this point it should be observed that if the hopper is just filled with clean water up to the coaming-level it only contains 3,240 tons, the total displacement (doors closed) is only 7,540 tons and the draught is only about 16 feet. Every additional foot of draught (doors closed) represents 500 tons so that an 18-foot draught requires another 1,000 tons to be added. If, in imagination, 1,667 tons of solid mineral (density 2·5) were to be added (volume 667 cubic metres), its volume would push 667 cubic metres of the water overboard, occupy its space and so provide the required additional 1,000 tons. Assuming, for simplicity in calculation, an in-situ density of 1·75, then 1,667 tons of mineral corresponds to the mineral content of about 1,333 cubic metres of mud or 2,333 tons of mud.

The mixture in this hypothetical case has a density of

$$\frac{(3{,}240 - 667) + 1{,}667}{3{,}240} = 1{\cdot}31.$$

It will be seen from this example that it is impossible to put the vessel down to the required draught even with a full hopper unless the pumped mixture has a certain rather high density. Conversely, if the mixture has more than this density, the hopper cannot be filled up to the coaming-level without exceeding the critical draught.

The Author may seem to have laboured this aspect of the matter, but it is a fact that certain dredger-masters appear to think that if they get full hoppers with a moderately thick mixture they are doing as well as possible, whereas the maximum output is obtained by getting maximum draught provided only a small percentage of extra pumping time is required.

Oblique Currents.

The tidal current on the Yangtse bar is rotary, the particles of water moving in a long ellipse, and the axis of this ellipse is inclined to the *thalweg* or valley line over the bar at an angle of about 40 degrees. Consequently at the time of full flood- or ebb-tide the current, which at spring tides may rise to almost 6 knots, is on part of the cut strongly athwart the axis of the cut. The dredger is therefore obliged to steam obliquely towards the current so that the combined velocity will give the proper direction. Only when the tides are weak is it possible to dredge with the current, as since the cutting speed rarely exceeds 3 knots there must be a margin for steering, and if there is any obliquity of current the triangle of velocities gives an impossible position.

When dredging in the stronger tides with a lateral "set," the mouth of the draghead is not square to the cut and a loss of entry-area occurs. In addition to this there is a much reduced reserve of thrust since the skin friction of the ship, and consequently the resistance to propulsion, are increased by the current. The vessel has a total of 3,000 indicated horse-power available in the propelling engines and can travel at about 11 knots in deep water with full hoppers, but even so it has been considered advisable to provide a larger percentage of thrust-reserve in the new ship. The pipe ladder is pressed strongly against the side of the well by the transverse reaction, and it has been found expedient to cut a hole in the side of the draghead to increase the influx of spoil.

This transverse current has been also the principal reason for adopting the centre-well type of dredger, which allows the vessel to be swung easily around the point of contact (the draghead). In the older Frühling vessels stern-wells were usual and have many advantages from the point of view of the arrangement of the machinery, but are much less suited to good steering in transverse currents.

Costs.

During the 2 years which the vessel has been operating the unit cost of dredging per cubic yard of in-situ material dredged and dumped about 2 sea miles from the cut has been about $0·20 Chinese currency, which was equivalent to 3*d*. This includes depreciation at 8 per cent. compound discount on the original cost of the ship (roughly £160,000), overhead charges, salaries, wages, stores, repairs, and insurance. It does not include any interest or the cost of survey work.

The second vessel will cost nearly 50 per cent. more owing to increased prices and the changes in design, so that the unit cost of dredging will also be increased slightly.

ACKNOWLEDGEMENTS.

The Author must admit his indebtedness in connexion with the design of the dredger to Mr. William Smith, Mr. P. N. Fawcett, M. Inst. C.E., and Dipl.-Ing. J. Kolkmann of Messrs. Schichau. His predecessor, Lt.-Col. A. W. H. von Heidenstam, M. Inst. C.E., and the late Sir Frederick Palmer, Past-President Inst. C.E., first directed his attention to many of the salient problems.

The Paper is accompanied by two sheets of drawings and six photographs, from some of which Plate 1 and the half-tone page plate have been prepared, and by the following Appendix.

APPENDIX.

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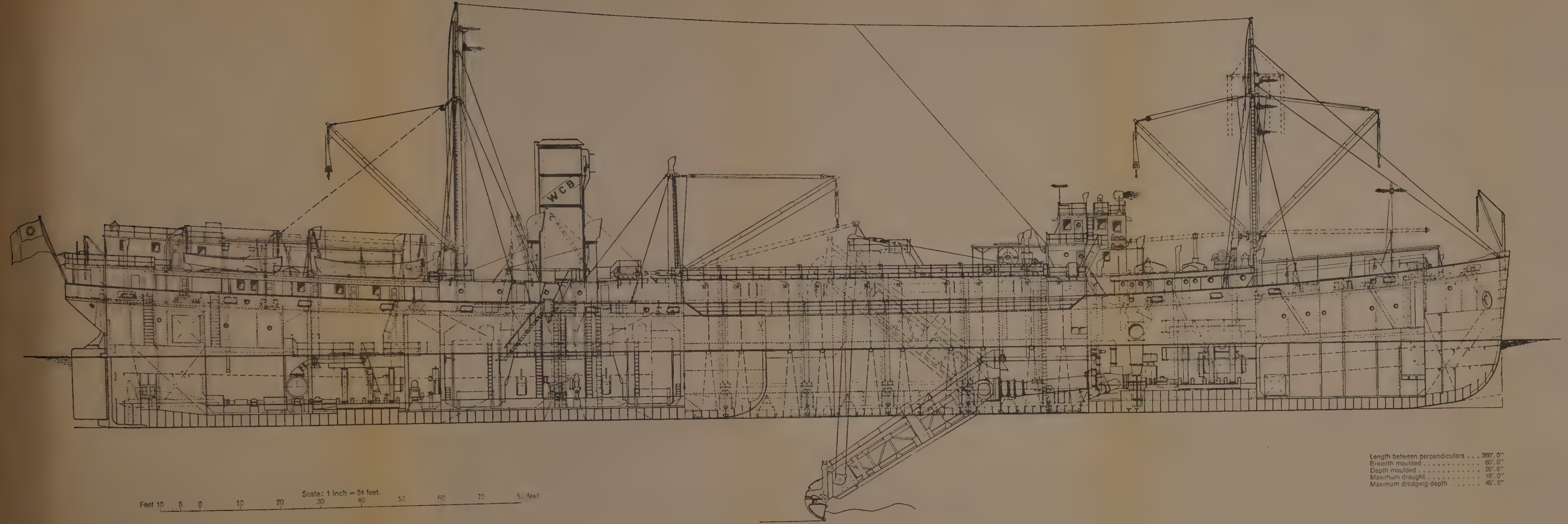
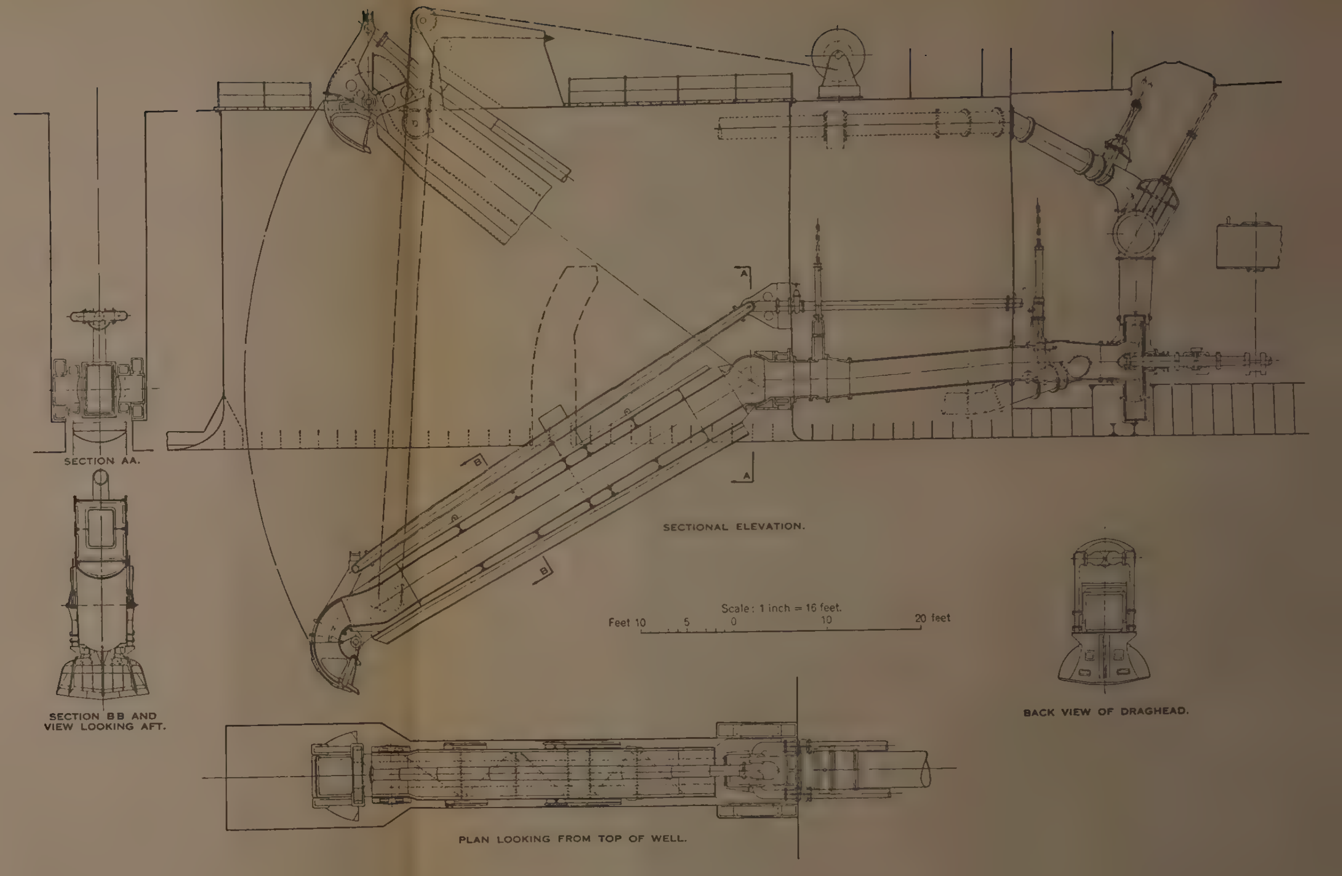
THE PRINCIPLES OF DRAG-SUCTION DREDGING.

Fig: 1.



THE TWIN-SCREW DRAG-SUCTION HOPPER DREDGER "CHIEN SHE."

Figs: 4.



ARRANGEMENT OF OUTBOARD SUCTION PIPE AND DRAGHEAD.

H. CHATLEY.

William Clowes & Sons, Limited: London.

Paper No. 5192.

"The Subterranean Sources of Water in the City of Rangoon."

By Herbert Cecil Edgar Cherry, M.Sc., M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

The Paper explains that water is raised from underground through tube wells for both domestic and industrial purposes, including the replenishment of the fresh-water tanks of ships in harbour. This supply represent 40 per cent. of that from catchment-reservoirs distributed by the local authority to 280,000 of the population of 400,000, because the latter only supplies water to about one-third of the main part of the city, which covers an area of 17 square miles, only a fraction of which is at present equipped with public sewerage. The relative importance of the subterranean supply raised a fear that the existing wells were causing mutual interference, were probably drawing on the underground reserve, and were producing at times unpotable water (especially from those wells sunk near the tidal rivers). In 1932, therefore, legislation was passed necessitating the registration of all tube wells, the furnishing to the Government of particulars with regard to them, and the need for obtaining a licence for the sinking of all future wells.

The administration of this Act has provided useful information regarding the sizes and depths of wells sunk, the strata through which they have passed, and the quantity and purity of the water extracted, and this information has been studied by the Author.

The topographical features of the city and its geology are described and illustrated in the Paper, with particular reference to the location, dimensions, and the capacity of the water-bearing strata. A chart is given showing the positions of the wells still working, and the relative reduced levels of the bottoms of the tubes. The water-tube is discussed, together with the effect of the tides thereon. An estimate has been made of the total quantity of water extracted, and of the amount of annual replacement from rainfall on the surface. The purposes for which the water is used are stated. Particular emphasis is given to the congestion of heavily-pumped wells in one corner of the city from which the bulk of the underground water is raised, and the chance of these wells becoming brackish.

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[1] Copies of the Paper may be obtained on loan from the Loan Library of The Institution; a limited number of copies is also available, for retention by members, on application to the Secretary.

Engineering details relating to the tube wells, and the methods of sinking and operating them, are given. Mention is made of the laboratory examination of samples of water taken from all wells at the time of registration, and it is pointed out that the results indicate the need for control in order to ensure that water taken for domestic use is potable. It is shown how these underground sources might become contaminated and irretrievably ruined by an influx of the saline water lying beneath the flat lands of the Irrawaddy river delta surrounding the city.

Conclusions are presented with regard to the probable success or otherwise of sinking wells in various regions of the city, the reserve storage being not yet drawn upon, and with regard to the probable continued value of an underground water-supply in spite of the fact that the municipal corporation are now constructing an additional large reservoir, which, when completed in 1942, will provide a piped supply of fresh water throughout the main portion of the city between the tidal rivers which practically embrace it. Mention is also made of the availability of underground supplies to the north of the city outside its boundaries, which, as now, will continue to be useful sources of supply to the extra-municipal suburbs which are rapidly developing along and between the three main roads leading out from the city in that direction. Suggestions are also made with regard to the places within the city-boundaries at which suitably-spaced wells could be advantageously sunk in the future, to the policy needed in that particular area in which large wells drawing vast volumes of water for commercial purposes are at present concentrated, and to whence the water that is now being drawn from these wells could be obtained should extraction in this quarter have to be curtailed or abandoned.

The Paper has been presented to The Institution with the permission of the Government of Burma.

The Paper is accompanied by three sheets of drawings.

NOTE.—The Institution as a body is not responsible either for the statements made, or for the opinions expressed, in the Papers published.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Committee on Wave Pressures.

In the November 1935 Journal [1] it was announced that a Committee had been formed to investigate the problems of wave pressures on sea structures, and proposed preliminary investigations were outlined. Those investigations were duly carried out, and it was reported in the December 1936 Journal [2] that they had indicated the value of small-scale model experiments and that a programme of such experiments had accordingly been drawn up in collaboration with the Department of Scientific and Industrial Research. The initial programme provided for 1 year's work, to be carried out by Dr. C. M. White, of the Imperial College of Science and Technology, London, at a cost of about £600, The Institution contributing £300. Major R. A. Bagnold subsequently took over the experimental work, under the supervision of Dr. White, and the programme was extended for a further year (1938–39). Major Bagnold has now prepared a comprehensive report on the experimental work so far carried out, which is printed below as an Interim Report of the Committee.

The Committee hope that it may be possible to arrange for the extension of the research for a further year to deal with some of the points requiring attention that are listed at the conclusion of the Report. The present personnel of the Committee is :—

Sir Leopold Savile, K.C.B. (*Chairman*).
A. L. Anderson, C.B.
Major R. A. Bagnold, M.A.
W. T. Halcrow.
R. E. Stradling, C.B., M.C., Ph.D., D.Sc., Director of the Building Research Station.
Assistant Professor C. M. White, B.Sc., Ph.D., of the Imperial College of Science and Technology.

The Committee wish to record their regret at the loss they have sustained by the deaths in 1938 of their fellow-members, Mr. G. G. Lynde and Mr. H. H. G. Mitchell, whose interest and support in the work of the Committee were greatly valued.

[1] Vol. 1 (1935–36), p. 43.
[2] Vol. 4 (1936–37), p. 310.

Interim Report on Wave-Pressure Research.

By Major Ralph Alger Bagnold, M.A.

TABLE OF CONTENTS.

Introduction.

The investigation to be described is an attempt to throw light on the nature of the shock pressures exerted on the face of a vertical sea-wall when a wave breaks against it.

The subject of wave pressures is discussed periodically at the International Navigation Congresses. As a result of the Brussels Congress of 1935 work was initiated in Great Britain by the Research Committee of The Institution of Civil Engineers. It was sponsored jointly by this Committee and by the Building Research Station, and was carried out under the supervision of Assistant Professor C. M. White, Ph.D., B.Sc., in the Civil Engineering Department of the Imperial College of Science and Technology, London. Experiments were started during the summer of 1937. The research, which has been confined to model-experiments, has been to some extent complementary to the elaborate full-scale measurements undertaken by the French Department of Ponts et Chaussées, with whom close touch has been maintained.

The main problem awaiting solution was as follows. It has been known for some years through the work of French and Italian experimenters[1] that two types of pressure are exerted by waves on sea-walls. In deep water the advancing wave does not break, but contact with the wall deflects the water upwards as a *clapoti*. In this case the pressure on the wall at any level is merely the hydrostatic head corresponding to the height of the top of the *clapoti*. The pressures are therefore small, but they last for periods of many seconds while the water is rising and falling. The mathematical theory of the *clapoti* has been worked out by Sainflou[2], and the

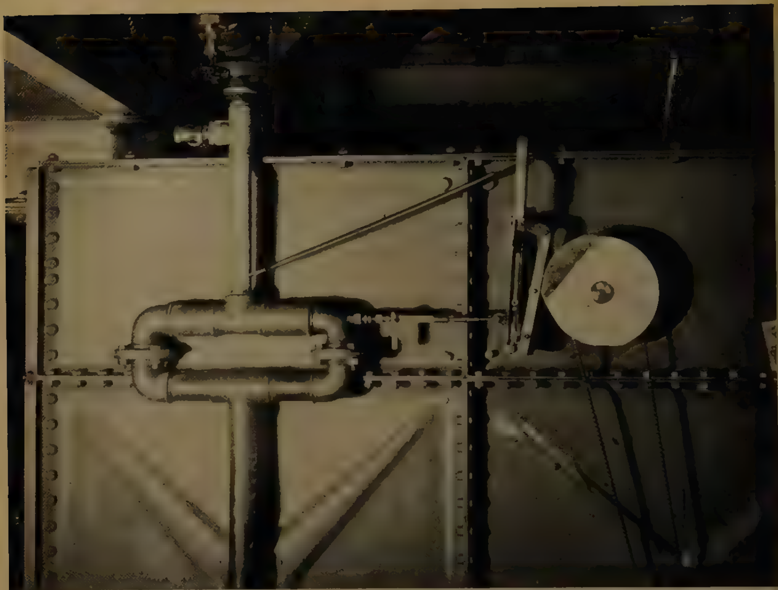
[1] Previous work is ably summarized in "Etat actuel des Etudes internationales sur les Efforts dus aux Lames", by A. de Rouville, P. Besson, and P. Pétry. Ann. Ponts et Chaussées, vol. 108 (II) (1938), p. 5.

[2] "Essai sur les digues maritimes verticales." Ann. Ponts et Chaussées, vol. 98 (II) (1928), p. 5.

Fig. 1.

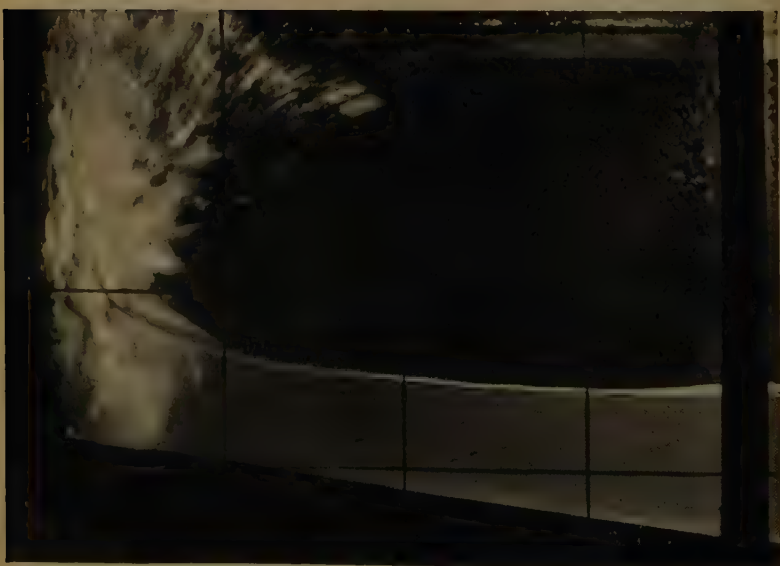
EXPERIMENTAL TANK.

Fig. 2.



WAVE-MAKING CONTROL-APPARATUS.

Fig. 4.



WAVE JUST AFTER IMPACT.

results have been satisfactorily verified both by French investigators at Dieppe, and by Italian investigators at Genoa. In localities where the water-level is constant—for example, in the Mediterranean—a sufficient depth of water is maintained at the wall to ensure that no impact other than that of the *clapoti* type can ever occur. The maximum pressures to be dealt with are therefore known as soon as reliable data have been collected regarding the maximum waves likely to be encountered.

Where the tide varies, however, the depth of water at the wall may be such that at low tide an oncoming wave may strike the wall at the moment of breaking, so that the advancing water face is nearly parallel with that of the wall. In this case, although the ultimate fate of the wave is the same as before—namely, it forms an upward jet which gives rise to a long-period pressure of low intensity—there also occurs, superimposed on the other, a shock pressure of great intensity but of very short duration.

Such shock pressures have been recorded at Dieppe up to a maximum value of 100 lb. per square inch. They occurred fitfully, however, and not more than 2 per cent. of the waves which struck the measuring apparatus gave any pressure at all. Not only has no useful correlation been obtained between the pressure maxima and the characteristics of the waves producing them, but no physical explanation has been forthcoming regarding how these pressures can arise. The problem is made more difficult by the entire absence of any theory or mathematical background for the mechanism of the breaking wave.

It was considered that the most fruitful method of studying such a problem was likely to be the use of a model wave-tank in which the characteristics of the wave causing the pressures could be maintained under close control.

The Model Wave-Tank.

The tank is shown in *Figs. 1, 2,* and *3.* It was made of standard pressed steel tank-plates 4 feet by 4 feet. The observation section, 12 feet long, was made of $\frac{3}{8}$-inch armour-plate glass in panels also 4 feet by 4 feet.

The wave-making apparatus consisted of a double-acting hydraulic ram A (*Figs. 3,* p. 204) working in trunnions and attached to a lever B pivoted at C. This had external and internal slides D and E, to which were attached connecting rods to the top and bottom of a plane steel paddle F whose weight was supported by two rollers G. The travel of the lever B and its speed-variation were controlled by a motor-driven cam shown in *Fig. 2,* which operated the valve gear. The travel of the top and bottom of the paddle could be adjusted independently by varying the heights of the two slides on the lever B.

The experiments were confined to a single wave driven forward along a level floor, made to break, by means of the sloping beach H, against the wall-face J, and reflected back again to meet the next stroke of the paddle, which meanwhile was held stationary. By altering the cam-setting and the

motor-speed two or more colliding waves could be produced, which travelled to and fro between the point or points of collision and the wall and the paddle. Since the water motion of the break (*déferlement*) seemed to be identical in both cases, however, there appeared to be no point in using anything but the one simple wave.

The wall consisted of two separate blocks of concrete of sufficient weight to withstand the maximum pressures by themselves without attachment to the tank sides. In the 4-inch slot between them a ½-inch stiffened steel plate, K, 15 inches high and 8 inches wide, carrying the piezo-electric unit L, could slide up and down. This plate was counterbalanced by the weight M, and was jammed in place by the lever and eccentric pivot N.

Figs. 3.

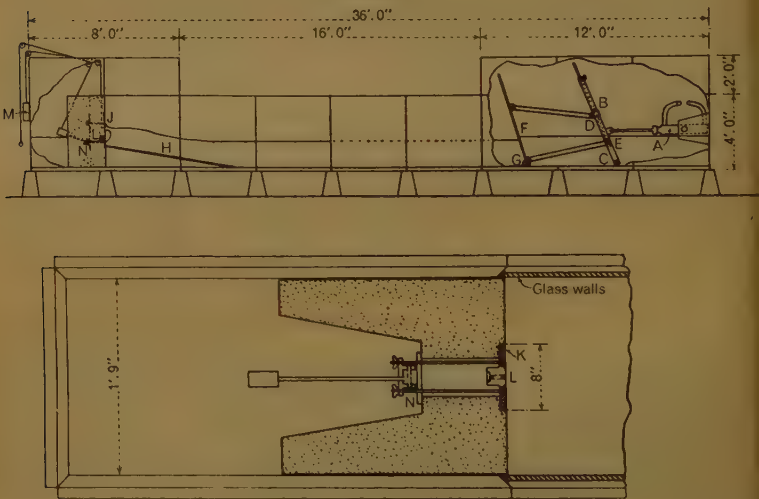


Fig. 4 (facing p. 203) shows a wave immediately after impact. The grid lines imposed on the glass tank-walls are 10 centimetres apart.

Pressure-Measurement.

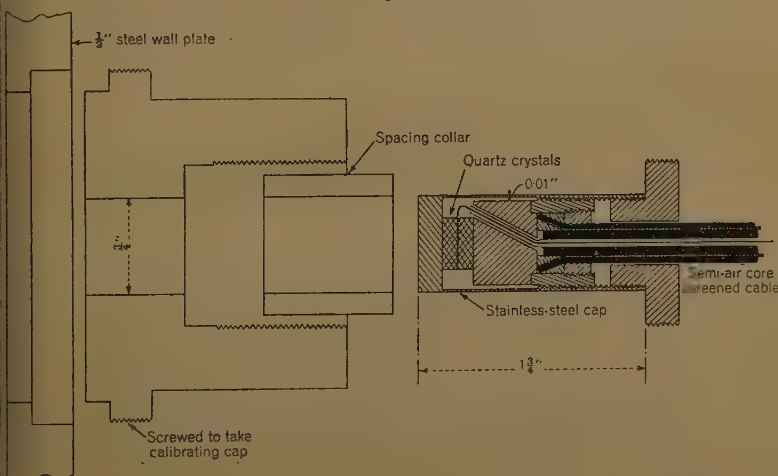
The piezo-electric unit L (*Figs. 3*) consisted of a stainless-steel capsule shown in detail in *Fig. 5*, containing a pair of quartz crystals. The capsule was mounted in a heavy brass plug. The electric charge produced by the mechanical compression of the crystals was conveyed to a two-valve amplifier through a screened cable of low capacity and high insulation resistance. The amplified impulses were fed through a second direct-current paraphase amplifier to a cathode-ray oscillograph, equipped with a time-base capable of giving any required spot-frequency. The most convenient spot-frequency was found to be 50 cycles per second. The whole

ressure-measuring apparatus was found to be practically aperiodic, and to e capable of response to shocks—produced by small direct metallic blows n the capsule face—of considerably shorter duration than any that were egistered by wave-impacts.

Calibration was effected by screwing a pneumatic cap, connected with a auge and pump, to the front of the unit. With the 12-centimetre cathode-ay tube used the most convenient amplification gave spot-deflexions of centimetre for each 10 lb. per square inch pressure.

After trials with various early forms of capsule and connexions little ifficulty was experienced in handling the extremely minute electric harges involved in the piezo-electric method of pressure-measurement, ven under the adverse watery conditions in which the work was carried out.

Fig. 5.



Since no satisfactory method could be found whereby the cathode spot ould be released automatically at the moment of wave-impact, photo-raphic records were taken by operating the camera by hand while watching he approaching wave. The only disadvantages of this simple method were i) the pressure-peaks might occur at any point along the horizontal sweep f the spot, and (ii) the spot made several blank sweeps during the exposure eriod and so produced a continuous and somewhat heavy trace along the xis of zero pressure.

Experimental Results.

The Breaking Wave.

The following general picture was obtained by means of visual observa-

tion, aided by a slow-motion cinematograph film of the types of in-shore water movement which may occur.

The Partial Break ("white horse").—A pure water wave of constant velocity and wave-length appears to exist only as a mathematical abstraction. Over a uniform horizontal bottom such a wave would travel unchanged in cross-sectional form. In nature a wave is composed of many components which may vary widely in wave-length and amplitude. It has long been known that when the amplitude of a wave exceeds about one-seventh of its wave-length the wave becomes unstable, and cannot continue without a change of form involving a dissipation of surplus energy. Since the velocity of a water wave depends on its wave-length phase-changes are continually occurring, and when two waves of large amplitude come into phase the above instability results in a "white horse." Observation shows that the excess energy is ejected outwards very suddenly from the combined summit as a thin jet of water (*Fig. 6*). The jet curls

Fig. 6.

over on one side or the other, and its energy is dissipated in a turbulent disturbance of the surface. If the wind-pressure is negligible, the side on which the jet collapses depends on its initial inclination, and this is always away from the component wave whose amplitude is the greater. The angle of the initial inclination from the vertical appears to vary with the ratio of the two amplitudes. This production of "white horses" is therefore not the direct result of wind-pressure, but is the mechanism whereby the excess of energy produced by the amalgamation of two waves is dissipated.

The curling-over of the jet containing the energy which the wave can no longer retain is called a "break." The "white horse" is a partial break. When the whole wave becomes unstable, however, its energy is, if space is available, dissipated altogether in a full break. A partial break may occur by the superposition of any two or more waves, whatever their respective wave-lengths may be, and a small parasitic wavelet may cause a partial break when it is raised to the summit of a larger wave of sufficient amplitude-ratio.

Reflexion from a Wall (*Clapoti*).—The ejection of excess energy when two waves meet occurs whether they are travelling in the same or in opposite directions with regard to the bottom. In the latter case, however, the sense of rotation of the water particles differs, so that the result is not quite the same, unless the two amplitudes are very different. When a wave is reflected from a vertical wall and meets its oncoming fellow, the

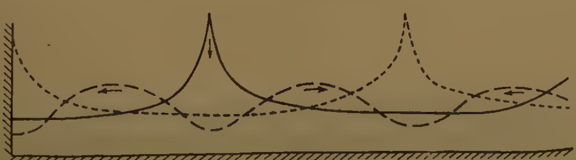
pposite sense of rotation of the water particles produces a more symnetrical jet. If the two waves are of nearly equal amplitude the jet rises ertically, and, being broader at the base, collapses on itself instead of urling over. Hence its energy is not dissipated in turbulence.

Two distinct wave-motions result, depending on the ratio of the wavength to the water-depth; that is, on whether the waves are "surface vaves" or "long waves" (shallow water). In the case of surface waves, vhen the water particles have a nearly circular motion, and the effect of he bottom can be neglected, reflexion produces true standing waves or *lapotis* (*Fig.* 7). Here the wave-peak has no horizontal motion, and the entre of curvature of the rest of the surface is always above the surface.

Fig. 7.

In the case of long waves, the collapse of the jet or *clapoti* forms two ravelling waves which move away in either direction from the site of the et, as shown in *Fig.* 8. The subsequent collision of pairs of these travelling

Fig. 8.



vaves gives rise to a second set of *clapotis*, and so on. At the wall, therefore, he effect is the same as if a succession of single travelling waves were to dvance against it.

Full Break on Beach.—When a travelling surface wave advances ufficiently far over a gently rising bottom, it tends to become a long wave vhen the water-depth is less than about a quarter of a wave-length. The elocity, which was originally a function of the wave-length only $\left(U = \sqrt{\frac{g\lambda}{2\pi}}\right)$, ecomes, when the depth is small, a function of the depth only ($U = \sqrt{gH}$).

The complete expression for the velocity in any depth of water is

$$U^2 = \frac{g\lambda}{2\pi} \tanh \frac{2\pi H}{\lambda},$$

neglecting the effects of surface-tension. Hence, as the bottom rises the wave is slowed up, and the wave-length is decreased. The energy o the wave is maintained by a change in the wave-form, the amplitude increasing to offset the reduction in velocity, until the limit of stability i reached at an amplitude/wave-length ratio of about 1/7.

If the slope of the bottom is steep (*Fig. 9*), however, the wave has n time to adjust either its velocity or its amplitude, both of which remai nearly constant (the velocity being taken as that of the summit). Th wave-front steepens very rapidly, till it becomes vertical. The wate below, which would, but for the bottom, have been pushed on ahead, i now scooped up the wave-front towards the top, as indicated by the arrows in the figure. The wave-energy again tends to be driven out in the form o a jet, but in this case, since there is no second wave, the jet is horizonta The jet advances at nearly twice the velocity of the wave summit, and fall to the beach where the whole wave finally dissipates itself in a series o vortices.

Defining the position of the break on the beach with regard to the stil water shore-line as that of an ordinate through the point where the wave

Fig. 9.

front first becomes vertical, this position depends on the beach-angle an on the characteristics of the original wave. Unfortunately the position i also very sensitive indeed to other factors which are hard to define. Th chief of these are (i) the amplitudes and phase-relations of the random wavelets which are present on the water-surface before the arrival of th wave; (ii) the strength of the current of water flowing down the beac from the wash of the last wave; and (iii) the surface-roughness and irregu larity of the beach.

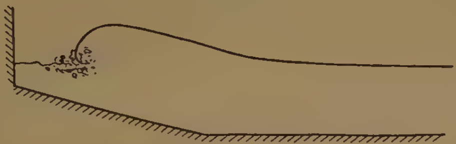
The first and most important of these factors, in addition to affectin the position of the break, also considerably complicates its action, for ver often a partial break will occur at the crest just before the main jet shoul otherwise develop. In view of the fact that no natural sea wave is eve free from these haphazard complications, it was not thought worth whil to attempt any quantitative work on the relations between the charac teristics of the main wave, the beach-angle, and the break position.

Full Break against a Wall.—If the bottom terminates in a vertical wal whose base is permanently submerged, the advancing wave, whether i

originates far away or results from a reflexion of the type shown in *Fig. 8* (p. 207), has three possible fates :—

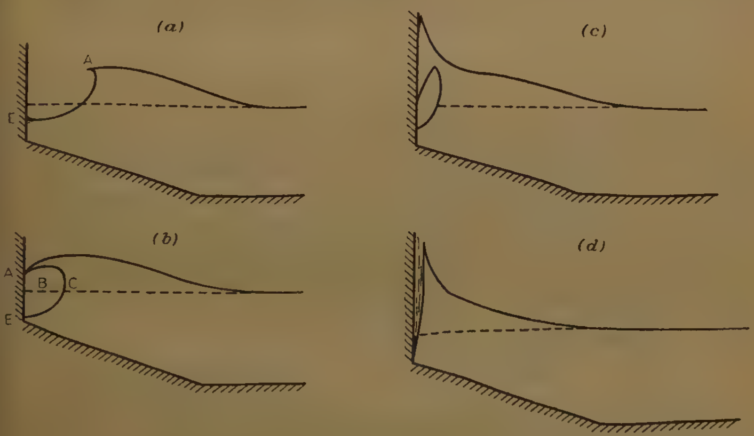
(i) The break may occur so early that the jet has collapsed before reaching the wall (*Fig. 10*).

Fig. 10.



(ii) If the break occurs later, the jet A (*Figs. 11* (*a*)) may strike the wall before it falls (*Figs. 11* (*b*)). In doing so it encloses a large cushion of air B between the wall and the lower part C of the wave-front. As this concave front advances, the air is compressed, and finally bursts upwards with a low booming sound and with the formation of much spray. The

Figs. 11.



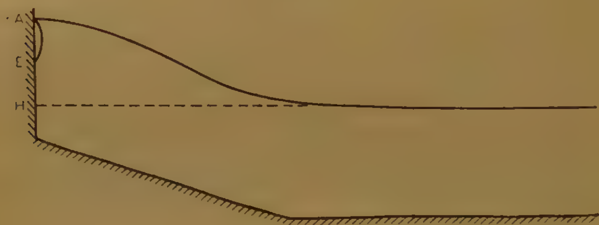
subsequent history of the break is shown in *Figs. 11* (*c*) and (*d*). It should be noticed here that the water-line E is lower than the general level of the surface in front of and behind the wave before it begins to break.

A still later break is shown in *Fig. 12* (p. 210). This is the only case in which the shock pressures appear ever to be produced. The air cushion is much thinner in the horizontal direction, and the water-line E has begun to rise from its lowest position before contact has been made at A. The

height of the cushion is therefore shorter in the vertical direction. The noise of the break is higher and sharper.

The velocity of rise of the water-line E is very great, and if the break happens a fraction of a second too late, E has reached the level of A before contact has been made. There is now no break at all, no air is enclosed

Fig. 12.



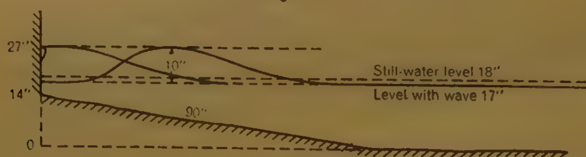
and there is no noise or shock. The water-line E runs steadily up the wall The wave has now become, in effect, one-half of a *clapoti.*

The condition necessary for the production of shock pressures, that is a very flat vertical wave-front, enclosing a thin cushion of air between itself and the wall, can only exist for a very short period of time. The condition is therefore extremely critical.

Shock Pressures Observed.

Shock pressures exceeding 10 times the long-period hydrostatic pressures had, as previously stated, been recorded by the French investigators at Dieppe, but attempts to correlate the pressure maxima with the observed characteristics of the waves causing them had failed completely. It was

Fig. 13.



hoped that in the model-tank, with wave-production under control, it would be possible to find this correlation, or at least to repeat at will shock pressures of constant maximum values, so that their distribution over the wall could be accurately explored. Accuracy of repetition being the first essential, attention has been concentrated on that form of wave which was found to give the most consistent results. The arrangement is shown in *Fig. 13.* A single wave was made to travel from the paddle to the wall,

where the break was caused by the slope of the beach. After reflexion the wave travelled back to the paddle, which had remained stationary the while, and which was timed to make another stroke at the right moment to drive the same wave forward once more. The wave-amplitude was 10 inches, the velocity of its summit was 6·8 feet per second, and the velocity of the vertical wave-front just before impact was 8 feet per second.

At the outset hydraulic power to the wave-making paddle was controlled by the speed of a direct-current motor driven off the mains. Although a breaking wave of constant amplitude could then be repeated indefinitely, the mains-voltage fluctuation of some 3 per cent. made it impossible to control the position and timing of the break well enough to maintain a succession of impacts which would all give measurable shock pressures. The shock pressures, as in the French work, occurred fitfully. In 90 per cent. of the impacts there were no shock pressures at all; the majority of the shock pressures when they did occur did not exceed 10 lb. per square inch, but very occasionally maxima as great as 35 lb. per square inch were observed.

A synchronous motor and a " Positive Infinitely Variable " gear for speed-control were then installed, and after a great deal of time had been spent in adjusting the wave-making apparatus, shock pressures could be obtained at every wave-impact. The necessary conditions were so critical that an alteration of but 0·3 per cent. in the paddle timing was sufficient to make the pressures disappear altogether.

The variation in the values of the pressure maxima from impact to impact was, however, still found to be as great as ever. Increasing the probability of their occurrence had only led to increasing the probability that still higher pressures would occasionally be observed. The highest pressure recorded was 80 lb. per square inch (visual observation only).

Typical oscillograms are reproduced in Figs. 14, Plate 1. Cathode-ray pictures of much higher pressure peaks have been observed visually, and examined in greater detail by the use of faster spot-frequencies. Reduced to the same time-scale as the photographs, such curves are typified in Fig. 15, Plate 1. These high pressures happened so seldom that unfortunately, in spite of several hundreds of exposures, no photograph of them has been secured.

From an examination of a large number of plates, and from the impression gained from the visual observation of a far larger number of cathode-ray pictures, when considered together with the simultaneous observation of the breaking wave itself, five conclusions have been drawn :—

(1) The shock pressures occur only when the shape of the advancing wave-front is such as to enclose an air cushion between it and the wall. The pressures are negligible when the thickness of this cushion outwards from the wall exceeds half its height, but they increase in intensity with decreasing thickness of the air cushion.

(2) The great variation in the pressure maxima from impact to impact,

even when to all appearances one wave is indistinguishable from another, must be due to a variation in the mean thickness of the air cushion arising from random irregularities in the relief of the concave water face as it meets the wall. These irregularities are inevitable. They are due to small parasitic wavelets, caused by the disturbances of the last wave-impact, of the paddle, and of partial breaks occurring at the wave crest during its journey from the paddle to the wall.

(3) Although the magnitude of the pressure-peaks varies enormously from impact to impact, the area enclosed by the pressure–time curves tends to approach, and never to exceed, a definite value. Thus a pressure which rises to a high peak value lasts for a shorter time than does a lower pressure.

(4) The horizontal zone along the wall on which the pressures were exerted was explored by sliding the steel wall-plate containing the piezo-capsule up and down the wall. Notwithstanding the difficulty in obtaining satisfactory repetitions of the phenomenon, it is very clear that both the high pressure-peaks and the maximum pressure–time areas only happen over the zone occupied by the air cushion. The height of this zone, between A and E in *Fig. 12* (p. 210), varied from 3 to 4 inches. Above A no shock pressures have been observed, and below E both the pressure maxima and the areas of the curves fell off rapidly with increasing depth. Near the bottom the oscillograms merged into those due to the long-period hydrostatic pressures.

(5) The maximum shock pressures ever observed in the model-tank have not exceeded one-sixth of the theoretical pressures possible if a true "water-hammer" were to occur. In the case of the French full-scale measurements the corresponding fraction is one-fourteenth. The full duration of the pressure in both cases always exceeds by 10 times, and generally by considerably more, the period required by the usual "water-hammer" theory. On this theory the duration of the pressure should be the time taken by a wave of compression to travel with the speed of sound in water (4,000 feet per second) from the seat of the impact to the nearest free surface at which the compression of the water can be relieved. On the other hand, as will appear later, all the observed facts seem to be consistent with the view that the energy of the impact in the case of the breaking wave is stored, not in the compression of the water, but in the compression of the air cushion.

An Explanation of the Observed Shock Pressures.

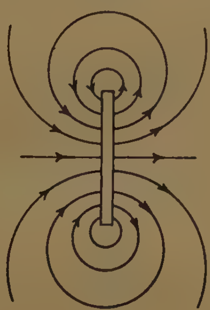
The "Kinetic Mass" of Water.

The physics of the production of short-period shock pressures by a breaking wave is not amenable to direct mathematical approach, because as long as the mathematical problem of the breaking wave itself remains

unsolved the initial conditions of the impact cannot be defined. The general principles which must be involved in the impact can, however, be stated fairly simply; then, by assuming empirical values for the dimensions of a certain real but indefinite volume of water, now to be discussed, useful quantitative results can be obtained.

The motion of a body through a fluid at rest necessitates motion also on the part of that portion of the fluid which surrounds the body. To an observer moving with the body, the movement of the surrounding fluid would appear as sketched in *Fig. 16*. Hence, in order to impart to the body a velocity u, kinetic energy must be given not only to the body, but to that volume of the fluid which is associated with the motion. The fluid velocity varies from point to point within this volume, and the volume is therefore indefinite in extent. It is convenient to imagine an equivalent volume in

Fig. 16.

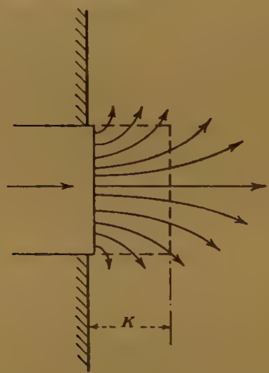


the form of a column of fluid of cross section A equal to that of the body, and of length K in the direction of the motion. This conventionalized fluid column is supposed to be such that if all the particles in it move with the velocity u of the body, the total kinetic energy remains the same as in the case of the real fluid. The associated fluid has, in fact, been replaced by a solid cylinder of the same density. Since this cylinder has a mass ρAK (which may be called the "kinetic mass"), it follows that, if the body be given an acceleration $\frac{du}{dt}$, an additional force $F = AK\frac{du}{dt}$ is required in order to accelerate the mass of the associated fluid. The fictitious length K depends both on the dimensions and on the shape of the body. For a sphere the associated volume of water is known to be half the volume of the sphere. If this volume is replaced by a cylinder of the same diameter d as the sphere, its length K is equal to $\frac{2d}{3}$.

If the body forms part of an otherwise stationary wall, as in *Fig. 17*,

and begins to move forward with an acceleration $\frac{du}{dt}$, there must likewise be an opposing force $\rho AK \frac{du}{dt}$ due to the inertia of the mass ρAK of associated water. Here K is some unknown length, which must, however, be

Fig. 17.



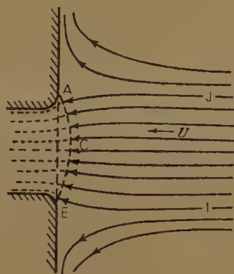
of the same order as the dimension of the accelerated face. It seems likely that K depends only on the boundary conditions, and remains constant for all accelerations as long as the displacement of the face remains inappreciable.

The kinetic mass ρAK, or rather the mass of the real but indefinite volume of water to which it is equivalent, exists as a separate entity, as distinct from the mass of the rest of the fluid, only by virtue of an acceleration of the boundary with which it is in contact. The boundary need not of necessity be the surface of a solid body; acceleration of the boundary, and therefore of the kinetic mass associated with it, can equally well be brought about if the solid body is replaced by a cushion of air at a higher hydrostatic pressure. Moreover, the kinetic mass must also appear in the reverse case when an advancing body of water is retarded.

The system shown in *Fig. 18* is that of steady flow against an infinite plate with a slot in it. It represents an idealized case of a breaking wave a very short time after the jet A of *Fig. 12* (p. 210) has made contact with the wall, and has enclosed a cushion of air whose outer surface is ACE. At the outset this cushion is at atmospheric pressure, and offers no resistance to the advancing water, which therefore behaves as if the slot AE were open. As the air is compressed, however, the force on the surface ACE due to the increasing air-pressure begins to displace the streamlines to the ultimate configuration shown in *Fig. 19*. Thus the retardation of the water is really the rate of displacement of the streamlines, and it is clear that the effect must be felt as a pressure on the wall beyond the limits of the air cushion.

The nature of the fiction of the "kinetic mass" is now apparent. For the extensive and indefinite volume in which the displacement occurs there has been substituted a cylinder of water bounded by the streamlines J and I projecting outwards from the surface ACE for a distance K. This cylinder is supposed to be solid, and to be advancing at the same rate as the mean

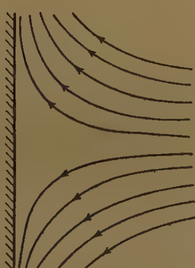
Fig. 18.



surface ACE. At the end of the compression the cylinder is brought to rest, and this is equivalent to the streamlines of the real water having reached their ultimate configuration.

If the length K can be found experimentally as an empirical function of some measurable quantity, the problem of the duration of the compression and the maximum pressure produced becomes, by the above artifice, the

Fig. 19.



straightforward problem of the retardation of a cylinder of water of unit cross section and length K which enters with initial velocity U the mouth of a cup of depth D containing air initially at atmospheric pressure (*Fig. 20*, p. 216).

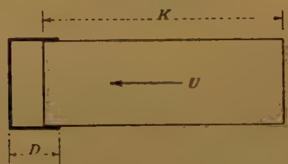
The Shock Momentum and the Length K.

The total forward momentum of the advancing wave is destroyed by the force of the wall acting for the long period during which the water is

rising to the top of the jet. Hence the time-integral of the force on unit length of the whole wall must clearly be equal to the total momentum of unit transverse length of the advancing wave. The time-integral of the force during the fall of the water jet must similarly correspond to the momentum of the reflected wave. Both of these forces are distributed over a great height and over a long period of time, however, and the pressure at any point is consequently small.

In the case of a breaking wave, when an air cushion is enclosed a small portion of the momentum of the wave is destroyed very rapidly owing to the instantaneous application of a large compressed-air force over a small but finite area of the water surface. The value of the momentum so destroyed must be ρUK per unit of superficial area of the air cushion, and this must be equal to the time-integral of the local pressure on the wall. The

Fig. 20.



value of this time-integral can be measured directly from the oscillograms, and the velocity of advance U is also known. Hence the length K can be found very simply:

$$K = \frac{\int p \, . \, dt}{\rho U}.$$

It has already been noted that the areas of the pressure–time curves obtained from the model-waves approach and never exceed a definite limit. This applies also to that portion of the curve between its beginning and its peak; that is, to the portion which, it seems reasonable to suppose, corresponds to the destruction of the original momentum of the kinetic mass of water involved in the impact. In the model-experiments, the area of this part of the curves gave a time-integral which approached a limit of 2·59 lb.-seconds per square foot. Hence, since the velocity U was 8 feet per second, a value of 0·166 foot, or 2 inches, is obtained for K. This is just half the observed height AE of the air cushion (and may be compared with the corresponding figure of two-thirds for a sphere). Since the height AE was found to be 0·4 times the wave-amplitude for those waves which gave appreciable shock pressures, it appears that the length K is empirically one-fifth of the wave-amplitude.

It is interesting at this point to apply the empirical value for K which the above ideas suggest to the case of the real waves whose impulses have

een recorded in the French report referred to on p. 202. Data for seven vaves are tabulated in columns 1 to 7 of Table I. The figures for the nomentum per unit area in column 7 are obtained by measuring the areas inder the compression portions of the pressure–time curves given in the eport. They are, therefore, entirely independent of any data other than he pressure actually measured on the sea-wall. The same momentum is rrived at in column 8 from the measured velocity and amplitude of the vaves, and from the empirical relation found from the model-work connectng the mean length K of the kinetic mass with the amplitude of the wave.

TABLE I.

Identification.				Wave Data.		From Model.	Shock momentum per unit area of wall: lbs./ft. sec.	
Col. (1) Date.	Col. (2) Time: hr.	min.	Col. (3) Height of instrument: metres above datum.	Col. (4) Amplitude, $2h$: feet.	Col. (5) Velocity, U: feet per second.	Col. (6) $K=2h/5$: feet.	Col. (7) $g\int_0^{\max.} p.dt$ (observed)	Col. (8) ρUK (calculated)
2/12/35 .	12	12	5·34	14·6	26·6	2·92	1,740	4,960
	12	15	4·37	14·6	26·6	2·92	2,050	4,960
4/12/35 .	14	08	4·37	9·75	19·5	1·95	3,050	2,440
	14	09	4·37	9·75	19·5	1·95	1,620	2,440
	14	50	4·37	9·75	19·5	1·95	2,150	2,440
18/12/35 .	13	35	4·37	13·0	23·0	2·6	3,400	3,840
23/2/37 .	12	37	4·37	8·12	22·0	1·62	2,420	2,260

In the French work attention was paid to the magnitude of the peak pressures rather than to that of the shock momenta, and the sample curves given in the report were presumably selected with the peak pressure in mind. It is the more satisfactory, therefore, that the shock momenta as derived from the pressure curves should show agreement with those which were derived from the wave-data by means of a relation found from model-experiments on one-twelfth the scale. It should be noted that the figures in column 8 should, according to the model-results, give the maximum impulse that a wave of a given amplitude and velocity can produce, whereas the figures in column 7 may range from zero up to this maximum according to the chance condition of the water surface at the moment of impact. It will be seen that the limit in column 8 has indeed been exceeded in two cases, and it is possible that the maximum value to be allowed for K may have to be made slightly greater than $2h/5$.

The Rise of Pressure. The Compression of the Air Cushion, and the Duration of the Impact.

Having arrived at an empirical method of calculating the length of the

horizontal water column involved in the impact on unit area of the wall, it is now possible to return to the consideration of the rate at which this unit column will compress various thicknesses of enclosed air, and at the same time to calculate the maximum pressure produced when the column is brought to rest.

The water column can be regarded as a solid plunger of length K, entering, at an initial velocity U, a cup of depth D containing air initially at atmospheric pressure p_0. If the distance from the bottom of the cup to the head of the plunger is denoted by x, then for the pressure of the enclosed air

$$p = Ax^{-\gamma}, \text{ where } A = p_0 D^{\gamma},$$

and for the motion of the plunger

$$p = \rho K \frac{d^2x}{dt^2} + p_0.$$

Hence the equation of motion of the plunger is

$$\rho K \frac{d^2x}{dt^2} - Ax^{-\gamma} + p_0 = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

On integration this gives

$$\rho K \left(\frac{dx}{dt}\right)^2 + \frac{2A}{\gamma - 1} x^{1-\gamma} + 2p_0 x = c.$$

When $x = D$, $dx/dt = U$. So, substituting the appropriate terms for c,

$$u^2 = \left(\frac{dx}{dt}\right)^2 = U^2 - \frac{2A}{(\gamma - 1)\rho K}\left[\left(\frac{1}{x}\right)^{(\gamma-1)} - \left(\frac{1}{D}\right)^{(\gamma-1)}\right] + \frac{2p_0}{\rho K}(D - x) \quad . \quad . \quad . \quad (2)$$

In the case of isothermal compression, where $\gamma = 1$, the corresponding equation is

$$u^2 = \left(\frac{dx}{dt}\right)^2 = U^2 - \frac{2A}{\rho K} \log \frac{D}{x} + \frac{2p_0}{\rho K}(D - x) \quad . \quad . \quad . \quad . \quad (2a)$$

Assuming that the compression is adiabatic, the time t is given by a second integration,

$$t = \sqrt{\frac{\rho K}{p_0}} \int \frac{dx}{\sqrt{\left\{\frac{\rho K}{p_0}U^2 - 5D^{1\cdot4}\left[\left(\frac{1}{x}\right)^{0\cdot4} - \left(\frac{1}{D}\right)^{0\cdot4}\right] + 2(D - x)\right\}}} \quad . \quad . \quad (3)$$

This second integration for t has not been achieved, but a number of typical pressure–time curves have been computed by graphical integration. Two families of these curves are given in Figs. 21 (a) and (b), Plate 1. In

ach set of curves both the velocity of advance U and the length K of the vater column have been kept constant, and the thickness D of the air ushion has been varied. In Figs. 21 (a), Plate 1, U and K correspond to the onditions in the model-tank, and in Figs. 21 (b), Plate 1, they correspond o the mean of the full-scale conditions in the French report.

Over the range considered from 2 to 10 atmospheres :—

(*a*) The peak pressure is given within $\pm$ 10 per cent. by

$$(p_{\max} - p_0) = 2{\cdot}7 \frac{\rho U^2 K}{D}$$

in any consistent units.

(*b*) The whole duration of the compression approximates, for high peak pressures, to the time taken by the water front to travel the distance D at the initial speed U, but for low pressures the duration is relatively shorter. If $T = a\frac{D}{U}$, a has a value of 1·1 for a peak pressure of 60 lb. per square inch, of 1·7 for a pressure of 20 lb. per square inch, and of 3 for a pressure of 7·5 lb. per square inch. This provides a simple means of estimating the thickness of the air cushion from the pressure curves.

(*c*) The area under the curve is constant as long as the product UK remains constant, no matter what may be the thickness D of the air cushion. This is obvious from the assumed simplified physics of the plunger and cup, for the value of $\int p \, . \, dt$ must be equal to the initial momentum ρUK under any conditions of compression. Under the actual conditions of wave-impact it is probable that the compression-index γ varies during the compression, and that rapid cooling may reduce γ to unity (isothermal compression) towards the end of the stroke. This will affect the shape of the curve, but not the value of $\int p \, . \, dt$.

It is likely, too, that there will be a preliminary small but gradual rise of pressure while the initial shape of the water front becomes modified by the enclosure of the air cushion. The fate of the air cushion during the compression is still not clear, as observation is difficult owing to the very rapid movement of the water surface. Immediately afterwards, however, the air appears in the form of small isolated bubbles. From this it seems likely that the water front becomes unstable during its retardation, and breaks up into little jets, as sketched in *Figs. 22* (p. 220). Since all such jets will certainly not strike the wall at the same instant, the pressure at any one point on the wall is likely to be built up in a series of steps as the pressures propagated from surrounding compressions reinforce that of the compression immediately over the recording instrument.

With these considerations in mind, the calculated pressure curves of Figs. 21 (a) and (b), Plate 1, may be compared with the actual oscillograms obtained from both model- and real waves. Those for the model are given in Figs. 14 and 15, Plate 1, and for the real waves some of the photographs given in the French report have been reproduced in Figs. 23, Plate 1. In each case the calculated curves have been drawn on the same relative scales

Figs. 22.

of pressure and time as the corresponding oscillograms. As far as the compression side is concerned, there is close agreement both as regards the general shape of the curves and the duration of the compression.

The Fall of Pressure.

If the compressive pressure is that required to absorb the momentum ρUK, then the subsequent expansion must recreate a corresponding outward momentum in the water.

It will be seen from the oscillograms that the fall of pressure after the peak has been reached is more irregular and more varied in form than the rise. The general shape of the curve strongly supports the view that an expansion of air follows the initial compression, and that the cycle is repeated as a damped oscillation. That this volumetric oscillation really occurs between the water and the enclosed air, and is not an instrumental effect, is borne out (*a*) by the large variation in its period, even though no part of the apparatus was altered during the series, and (*b*) by the fact that the same general pattern is seen in the French curves, where the time-scale is 10 times as long.

In addition to this phenomenon, however, there are many instances in which a sharp initial rise of pressure is followed by a second longer period during which high pressure is maintained. These cases, it should be noted, only occur when the time-integral for the initial pressure-rise is small compared with the momentum limit set by the dimensions of the wave. It seems reasonable to suppose that in these cases a small local jet of water strikes the recording instrument first, enclosing but a thin air cushion, and producing a short rapid pressure-rise. This is followed by the main com-

ression, which occurs over a much larger area simultaneously, and there-ore involves a greater kinetic mass of water. If this main compression ncloses a thick air cushion (ratio K/D approaching unity), and if its peak ccurs during the expansion of the earlier local shock, the effect is likely to e a pressure picture such as No. 10 of Figs. 14, Plate 1.

It is therefore difficult to draw the dividing line between compression nd expansion; the net result may be that due to pressures rising and alling in different areas at different times. From the practical stand-oint it is probably more convenient to consider only the total impulse, as epresented by the area under the whole pressure–time curve. It appears rom the curves that the maximum value of this total impulse is given by nerely doubling the value of ρUK.

robability of the Occurrence of High Shock Pressures.

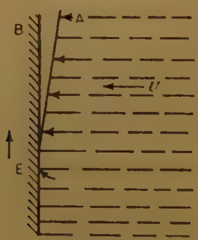
Under the idealized conditions in which a single continuous cushion of ir is evenly compressed by a smooth face of advancing water, the prob-bility of the occurrence of a pressure peak exceeding any given value an be estimated at once: it is only necessary to postulate a varying uccession of waves such that there are equal chances that the thickness of he air cushion enclosed has any random value between $D = 0$ and $D = K$. Then since for a constant wave-velocity U the value of the pressure peak aries as K/D, the chances of any pressure higher than that corresponding o any particular value of D are simply 1 to K/D. For example, in Figs. 21 (a), Plate 1, the value of K/D which gives a pressure of 36 lb. per quare inch is 16·6, so that the chances that a pressure exceeding 36 lb. per square inch will occur should be 1 to 16·6. Actually, however, obser-vation shows that the chances are very much smaller, and the reason is not ar to seek: the initial assumption of a continuous smooth advancing water face can rarely, if ever, be fulfilled. The water surface must always contain irregularities and prominences. Suppose that in the above uccession of waves the distance by which these prominences project in ront of the general surface is denoted by s. Then it is clear that the air cushion can never be continuous if its initial thickness D is less than s. In the above example, for instance, where the mean value of D is $\frac{1}{8}$ inch, nstantaneous compression will not occur evenly over the whole area covered by the cushion unless the prominences on the water face are less than $\frac{1}{8}$ inch high. Thus for the very small values of D which alone can give rise to high shock pressures, the cushion must in general be split up into a number of isolated air pockets, the compression of which will very rarely occur simultaneously. Hence the compression of one such pocket must in general be relieved by a sideways movement of the pocket along the wall into a region of lower pressure where the compression has not yet proceeded so far.

This raises the paradox that under atmospheric conditions, if the air

cushion is absent, or if the air in it is allowed to escape, the shock pressure, far from becoming more violent, disappears altogether.

The explanation seems to lie in the fact that, whereas in the case of the compression of the air cushion pressure is applied simultaneously over a finite water surface, in the latter case it is applied successively over different filaments of that surface. *Fig. 24* is an enlarged section of a water–air–solid junction at the edge of a real or potential air pocket. The water surface above the contact-line E is supposed to be advancing unchecked with velocity U, and the water particles below E and close to the wall can have no horizontal velocity. The wedge of air ABE is pushed upwards, supposedly without offering any appreciable back-pressure to the movement of the water. The water surface AE may be imagined to consist of an infinite number of parallel filaments perpendicular to the

Fig. 24.



plane of the paper. As each filament comes successively into contact with the wall at E the momentum of the water in it and behind it is destroyed, and the water particles are deflected upwards. At the moment of the impact, however, there is no pressure above E, neither is there any pressure below E other than a small hydrostatic pressure. Hence at any one instant of time a shock pressure can only occur along the actual contact-line E. Further, since (a) the pressure there is in any case limited to the finite value set by the known true water-hammer pressure corresponding to the velocity U, and (b) the pressure occurs at any one instant of time only over the infinitesimal width of one filament, the force on the wall at E at any time must itself be infinitesimal.

This appears to hold good provided that the successive time-instants do not overlap; that is, provided that the upward velocity of the water-line E is less than the velocity c_w of the propagation of sound in water. That the upward velocity could ever exceed this value is highly improbable, for even if the presence of the wedge of air is neglected altogether, and vacuum-conditions are assumed, the velocity of E could only exceed c_w if the angle AEB in *Fig. 24* were less than $\tan^{-1} U/c_w$: for a practical value of 20 feet per second for U, the critical angle is found to be only

8 minutes of arc. In a vacuum, therefore, true water-hammer pressures hould ensue over a finite area, if the water surface is very nearly parallel o the wall. The presence of atmospheric air must make a considerable ifference, however, even though it be free to escape upwards, for the elocity c_a of the propagation of a displacement in air is only a quarter of hat in water. Hence, for angles AEB of less than 4 times the above value, he air in the wedge could not get away without checking the advance of he water by the exertion of a back-pressure. It therefore appears that nder atmospheric conditions true water-hammer pressures can never occur n a free unrestricted impact between a body of water and a solid, because here must always be an air cushion interposed.

On the other hand, the presence of air may entirely alter the argument hat with contact-angles greater than $\tan^{-1} U/c_w$ no finite shock pressure can occur anywhere; for, once the air becomes compressed, either by ecoming enclosed or by being unable to get away in time (when the angle EB is less than $\tan^{-1} U/c_a$), the instantaneous pressure on the wall, nstead of being limited to an infinitesimal area at E, is distributed over the hole surface in contact with the air. As a result, there is a sudden estruction of water momentum over a finite and large area, and a conequent large compressive force.

The above ideas are crudely expressed, but a rigid treatment is impossible in the absence of any mathematical background defining the true otion of the water.

There remains the question of why the maximum shock pressures ecorded for full-scale waves are relatively so small compared with those bserved in the model-experiments. The peak pressures, assuming diabatic compression, are given approximately by

$$(p_{\max} - p_0) = 2{\cdot}7 \frac{\rho U^2 K}{D}.$$

K appears to be proportional to the wave-amplitude, and U^2 is pproximately proportional to the water-depth. Hence, assuming the ame probability modulus, it would be expected that if the wave-shape is imilar the peak pressures observed should be proportional to the general inear scale of the phenomena. Since the model-pressures reached a aximum value of 80 lb. per square inch and the linear-scale ratio was 12, ull-scale pressures of 1,000 lb. per square inch ought to have been encountered, whereas the highest pressure peak recorded by the French is nly 100 lb. per square inch.

Whilst much of the discrepancy could no doubt be explained by the maller number of waves of the required shape and timing observed in the ncontrolled full-scale conditions under which the French were obliged o work, yet the main cause is probably to be found in the very different hysical properties of the surfaces of sea-water and fresh water. In sea-

water the permanence of air–water emulsions, and of froths and foam, i far greater than in the case of fresh water, and in the violently-disturbe water near a sea wall the volume of cellular air associated with the breaker is by common experience very large. This air must of itself form an effec- tive cushion, whether or not any additional continuous cushion is als enclosed between the wave front and the wall.

The volume needed to prevent any high pressure peaks from occurrin is quite small. With the typical values of U and K taken for the pressur curves of Figs. 21 (b), Plate 1, the value of D corresponding to a pressur peak of 100 lb. per square inch is only 0·36 foot, or 4·3 inches, and it is no unreasonable to suppose that the equivalent thickness of the air enclose in the internal emulsion and in the external foam is rarely much less tha this. Unfortunately, no figures are available.

If this reasoning is correct, the ultimate limit to the intensity of th shock pressures exerted on a sea wall is set by the quantity of air locked i and on the surface of the wave before impact. Since the foam-formin properties of the water are very sensitive to its organic and mineral content it is likely that the ultimate pressure-limit varies greatly with the geo graphical locality.

General Conclusions Concerning Shock Pressures.

(1) The shock pressure exerted by a breaking wave is due to the violen simultaneous retardation of a certain limited mass of water which is brough to rest by the action of a thin cushion of air, which in the process become compressed by the advancing wave front. The volume of water concerne is approximately equal to that of a horizontal column whose cross sectio parallel to the wall is that of the frontal aspect of the air cushion, and whos length K is half the vertical width of the cushion. Shock pressures som 10 times greater than the ordinary hydrostatic wave-pressure can b generated in this way.

(2) The pressure set up is determined by the initial velocity of approac U of this water column, by its length K, and by the mean initial thicknes D of the air cushion. The pressure at any moment during the impact ca readily be calculated from these three quantities by regarding the wate column as a heavy free piston which compresses the air cushion adiabatic ally. The maximum pressure is given approximately by

$$(p_{\max} - p_0) = 2{\cdot}7\rho U^2 \frac{K}{D}.$$

(3) The greatest pressure-maxima therefore occur only when the thick ness D of the air cushion is small, and when compression is simultaneou over a large area of this thin cushion. The wave front must thus b approximately plane and parallel to the wall at the moment of impact.

(4) Waves originally of many quite different forms can conform to this ondition, but the maximum vertical width of a thin cushion of enclosed ir is of the order of 0·4 times the wave-amplitude $2h$. Hence the maximum alue of K is, by (1), equal to 0·2 times $2h$, and the maximum pressure-rise an be written

$$(p_{\max} - p_0) = 0{\cdot}54\rho U^2 \frac{2h}{D},$$

nd so, for any known wave-amplitude and velocity, varies inversely as the hickness of the air cushion.

(5) This thickness D is indefinite, since it is sensitive to the random regularities of the surface of the advancing wave front, so that maximum ressures cannot easily be calculated. It seems probable that in the very isturbed water inevitably found near sea walls during storms, enough air s entrained or held on the surface as foam to provide in itself a lower limit o the possible values of D, and that a limit is thereby set to the maximum hock pressure which can be exerted. This, however, could only be estabished by full-scale measurements.

(6) On the other hand, the impulse I has a definite maximum which is ndependent of the air-thickness D, and can be predicted from values dislosed by the model. Per unit area of the wall the impulse is simply

$$I = \int p \, . \, dt = 2\rho UK,$$

nd this has the maximum value $0{\cdot}8\rho Uh$, where $2h$ denotes the wavemplitude, and U its velocity. The impulse is thus equal to twice the initial nomentum ρK of the "kinetic mass" of water. When maximum impulses re compared, full-scale measurements at Dieppe are consistent with those nade on the model.

(7) The main pressure-zone on the wall extends only over the area overed by the air cushion. Below the bottom of the cushion the pressures ecrease rapidly. In the model the cushions which gave high shock ressures extended from the wave top, at a height $2h$, to a height $1{\cdot}2h$ bove the wave trough. The full-scale results from Dieppe seem to indicate somewhat lower level for the pressure-zone, but the wall was not vertical, nd it was impossible to determine the exact cross section of the waves at npact.

Suggested Lines for Further Research.

aboratory Work with the Present Model Wave-tank.

(i) Further pressure-measurements on the previous lines but with eflected travelling waves of the type shown in *Fig. 8* (p. 207), and with arying beach conditions, to verify that no larger shock impulses than those lready found are likely to occur.

(ii) Experiments on ribbing the outer wall surface to reduce the instantaneous pressure area and the resulting shock momentum.

(iii) The mechanical effect of short-period impulses of high peak pressure in moving concrete blocks which are variously loaded with superstructure.

(iv) The flotation-effects of the long-period hydrostatic pressures due to the rising *clapoti* when such pressures are allowed to penetrate into ill-fitting joints; the degree of close fitting necessary to prevent possible flotation.

(v) Experiments with loose beaches of sand and shingle to ascertain the effects of the bottom-movement of breaking waves in building up and removing the beach material. This might, in addition, provide useful information on coast-erosion.

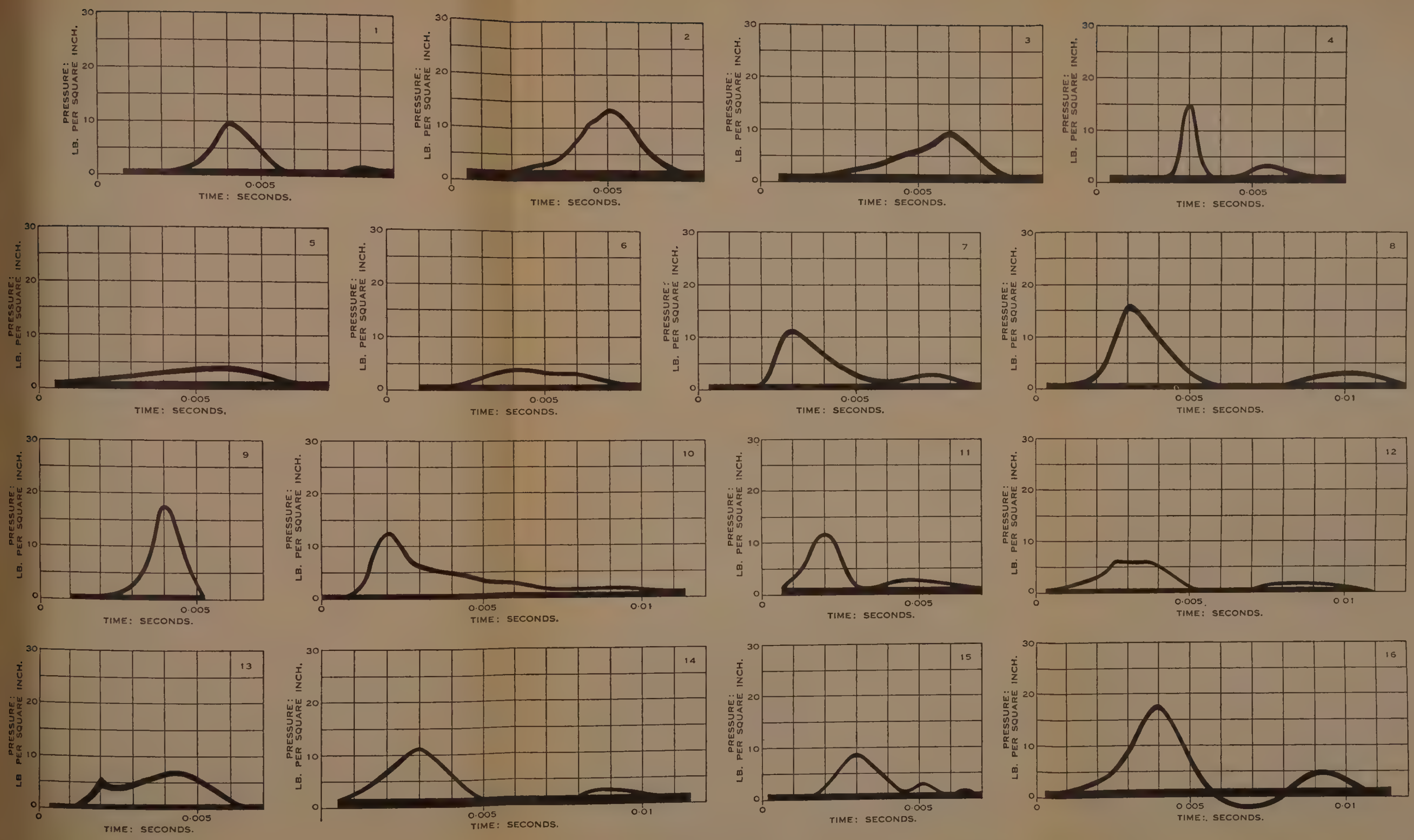
Full-Scale Work.

(i) The collection of information on the minimum air-content of breaking sea waves in different localities; the effect of differences in the organic and mineral make-up of the water on the permanence and volume of the foam and air-emulsion.

(ii) Pressure-measurements on sea walls in confirmation of the French results, with special reference to the position and extent of the pressure-zone.

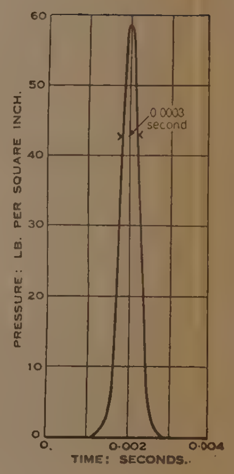
(iii) Verification of any useful results which may be suggested by the above laboratory work.

FIGS: 14.



MODEL-WAVE. PRESSURE IMPULSES PHOTOGRAPHICALLY RECORDED.

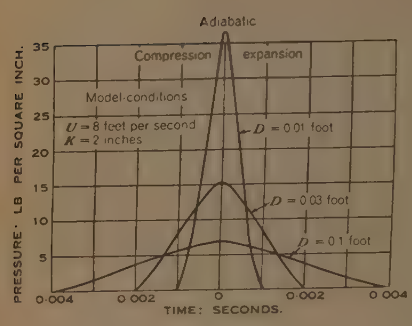
FIG: 15.



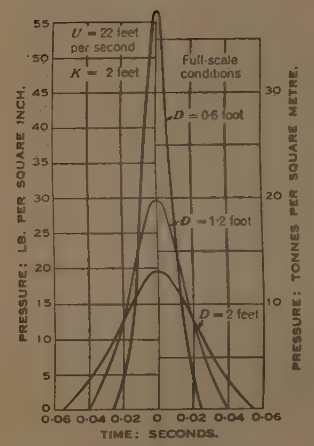
MODEL-WAVE.
RARE HIGH-PRESSURE IMPULSES AS OBSERVED VISUALLY.

FIGS. 21.

(a)

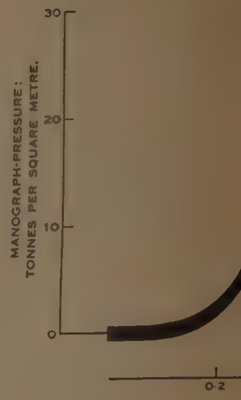


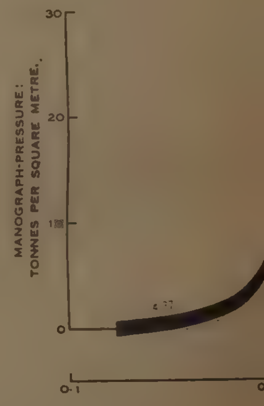
(b)



(a) MODEL-CONDITIONS (TO BE COMPARED WITH OSCILLOGRAMS IN FIGS: 14 & 15).
(b) MEAN FULL-SCALE CONDITIONS OF WAVES AT DIEPPE (TO BE COMPARED WITH OSCILLOGRAMS IN FIGS: 23).

PRESSURE IMPULSES AS CALCULATED FROM WAVE DATA, ASSUMING ADIABATIC COMPRESSION OF THE ENCLOSED AIR CUSHION.





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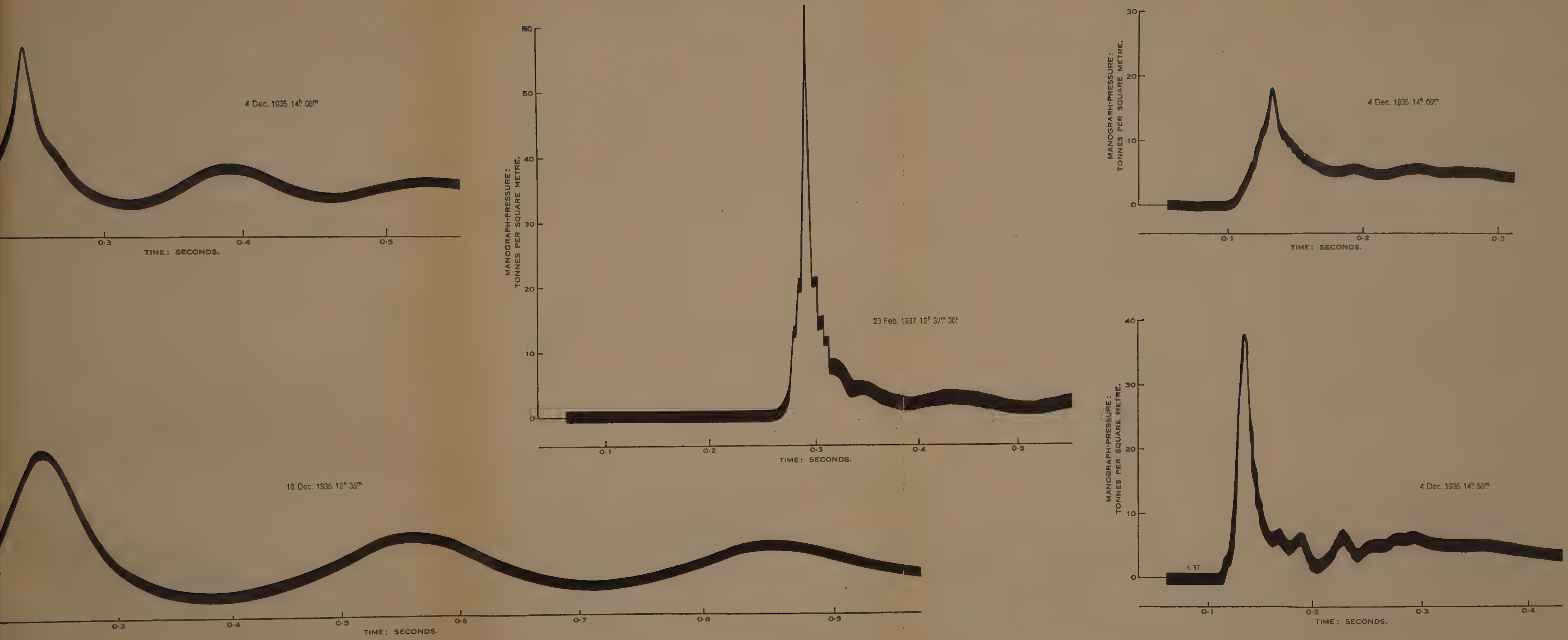
FIGS: 23.



FULL-SCALE SEA WAVES. PRESSURE IMPULSES AS RECORDED AT DIEPPE.

R. A. BAGNOLD.

REPORT OF THE NATIONAL PHYSICAL LABORATORY FOR THE YEAR 1938 *.

The following notes outline briefly some of the researches at present in progress in the Laboratory, which are described in the Report.

In the Physics Department, progress has been made on the maintenance of the International Temperature Scale, and results have been found to be in close agreement with determinations of the National Bureau of Standards, Washington. A practical method has been devised for measuring the temperature of liquid steel. The thermal properties of a number of carbon and alloy steels have been determined up to high temperatures. The application of X-ray diffraction to industrial and other problems has been extended, including the investigation of the structural changes associated with the fatigue failure of metals and of the effects of cold-working. Investigation of various problems in architectural acoustics has been continued, and many problems of noise-reduction have been studied. The study of the surface finish of metals has been continued.

In the Electricity Department, improvements have been made in the methods and equipment used in high-frequency measurements of the capacitance and power-factor of dielectrics. The study of the fundamental dielectric properties of insulating materials has been continued, and attempts have been made to connect the properties with the structure of pure materials of known chemical composition. Various problems of high-voltage power-transmission are being studied, and facilities for switchgear-testing by the Association of Short-Circuit Testing Authorities have now been made available. Research on illumination has included a series of experiments undertaken for the Ministry of Transport to determine the contrast between objects and the background necessary for safe driving at a speed of 30 miles per hour.

The work of the Radio Department has included the development of a constant-frequency oscillator, independent of outside control by mechanical or other means; this has been achieved by the design of inductances and condensers unaffected by temperature-change. The study of the source and characteristics of atmospherics has now been brought to a conclusion. Considerable attention has been given to the production and propagation of ultra-short waves, and the improvement of short-wave direction-finding has been continued.

In addition to the routine work of the Metrology Department, an interesting investigation has been carried out to explore the possibility of ascertaining the average temperature of steel survey tapes by observations of their electrical resistance.

The long-range investigations on the strength of metals carried out for

* Published by H.M. Stationery Office, price 2*s.* 6*d.*

many years by the Engineering Department have been continued. The work on the behaviour of metals under combined bending and torsional fatigue stresses has been advanced, and correlation has been obtained between the results of combined-stress fatigue tests with the fatigue limits for simple bending stresses and simple torsional stresses. The importance of such combined stresses, particularly in aircraft-engine crankshafts, has led to the use of larger specimens for the examination of complex conditions of stress. Progress has been made with the fundamental investigation on the deformation and fracture of metals, using X-ray diffraction methods to follow the internal changes of structure. The suitability of welding processes for pressure-vessels to contain liquefied gases is being investigated. The study of the strength and causes of failure of chains and components of lifting gear, which has been in progress for many years, has now been brought to a conclusion. Problems of ventilation and lubrication have also received attention.

In the Metallurgy Department, much work has been done on aluminium and magnesium alloys, including studies of the age-hardening properties of aluminium-copper alloys and of suitable forging conditions for magnesium alloys. Further work has been done on intercrystalline cracking and the caustic cracking of boiler plates; a definite theory on the cause of the latter phenomenon has now been formulated, and it is hoped shortly to terminate this research. Work is being continued on the creep of metals at high temperatures, both from the fundamental point of view and to provide data relating to the alloy steels used in modern steam power-plant; the Electrical Research Association is collaborating in the latter work. The study of refractory materials has been continued, and the technique of standardizing Seger cones has been improved.

Owing to the heavy demands on the Aerodynamics Department, two additional wind-tunnels are being built. Much of the recent work has been on models of new aircraft, and elaborate tests have often been required. The behaviour of land flaps and trimming devices has been studied, and the research on stability and control has been continued. A magnetostriction method for measuring the periodic forces on an oscillating aerofoil has been developed, and is being applied to the measurement of the pitching moment of an aerofoil. The theoretical investigation of flutter and buffeting is proceeding.

A reduction in the amount of routine testing has enabled more rapid progress to be made with the research programme of the William Froude Laboratory. Amongst the many problems that have been studied are the design of propellers to give a relatively high ratio of revolutions to speed of propulsion, the effects of varying the vertical positions of twin screws, the effect on resistance of the degree of smoothness or roughness on ships' plating, and problems of hull and propeller-blade vibration. A further stage in the research on the resistance, propulsion, and pitching of ships in rough water has been completed, and the general conclusion has been reached that the present method of designing screws on data obtained in

mooth water is sound in principle. Research on the design of various ypes of coasters has been continued. Full-scale investigations have icluded a complete series of turning trials on a new twin-screw vessel, nd some interesting effects were recorded.

REPORT OF THE BUILDING RESEARCH BOARD FOR THE YEAR 1938 *.

Special attention is given in this Report to the work recently carried ut on natural and artificial pozzuolanas. The valuable properties of ozzuolanas in protecting cement mortars and concretes from attack by ılphate salts are explained, and it is also pointed out that, owing to their fect in retarding heat-evolution, they offer an alternative material to low-eat Portland cement for use in large dams ; a pozzuolanic cement has been mployed in one large dam recently constructed in America. The pre-autions necessary for the effective use of pozzuolanas are explained, and iethods of tests have recently been developed which enable the quality f a pozzuolana to be determined. Artificial pozzuolanas can now be pro-uced in Great Britain from certain clays and from the spent material of ıe Scottish shale oil industry.

The Building Research Station carries out investigations on building ıaterials of all types, amongst which may be mentioned stone, tiles, sand–me bricks, asphaltic and bituminous roofing and waterproofing com-ositions, cast concrete products, limes and plasters, external rendered nishes, and paints. The resistance of brickwork to rain-penetration is ceiving careful study by means of a small rain-machine which simulates ıy desired rainfall with an accompanying wind of 30 miles per hour. The ıemical stability of fired clays is being studied.

Several important studies relating to cements are in progress. Arrange-ıents have been made for a large-scale investigation, in collaboration with he Institution, on the deterioration of concrete in soils and ground waters ntaining sulphate salts. The constitution of cement has been studied 7 the preparation of clinkers of different types under carefully-controlled nditions, and the differences in the properties of the cements resulting e discussed in the Report. In particular, it was found that cements oduced from a glassy clinker had a considerably higher resistance to tack by sulphate salts than those produced from a crystalline clinker. ethods for the determination of free lime in cement and cement-clinker e being studied, in co-operation with the American Society for Testing aterials ; it is now possible to attain reasonable uniformity in the deter-inations made by different laboratories and by different methods. Other oblems in the mechanism of the setting of cement have also received tention. Methods for the determinations of the fineness of Portland ment have been improved.

* Published by H.M. Stationery Office, price 3*s*. 6*d*.

The investigation on the use of foamed blast-furnace slag as an aggregat for concrete is practically complete, and attention is now being directe mainly to air-cooled slag for use both in slag macadam and in concrete.

The action of sea-water on reinforced concrete is being studied in colla boration with the Committee of The Institution on the Deterioration o Structures Exposed to Sea-Action, and particulars are given in the Repor of the condition of the numerous test piles which are under observation Generally speaking, it appears that a 2-inch cover is advisable, even wit rich concretes.

Among the investigations on structures and on the strength of material those on the strength and deformation of reinforced-concrete slabs and o the behaviour of eccentrically loaded long columns are of interest. Th latter investigation appears so far to show that the relevant requirement of the Code of Practice for the Use of Reinforced Concrete in Building are conservative ; the tests are being continued.

Progress is being made with the development of a hydraulic helme and a peak-stress indicator for the driving of reinforced-concrete piles this work is being carried out in collaboration with The Institution Another co-operative research with The Institution is the study of th compaction of concrete by vibration ; in general it may be said tha improved properties may be obtained in vibrated concretes owing to th lower water-contents that can be used. Instruments for measuring th maximum acceleration and amplitude of vibration in concrete are describe in the Report.

In connexion with the proposed revision of the British Standard Specifi cation for Portland Cement, extensive tests have been carried out on th correlation of tensile and compressive strengths. Discrepancies betwee the results obtained by various laboratories appear to be due mainl to differences in the degree of compaction attained by individual operator and compaction by a standard method of vibration is therefore suggeste Preliminary notes are given of a research at present in progress on th fatigue of cement mortars.

Tests have been undertaken in connexion with the preparation of British Standard Specification for Load-Bearing Concrete, Brickwork, an Masonry, to determine the permissible pressures on concrete and th reduction-factors for various slenderness-ratios. The work on reinforce brick beams has been extended to determine the utility of simple shea reinforcement ; it has been found to increase the strength of beams b over 50 per cent. The possibility of corrosion of the reinforcement in suc beams is also being studied.

Details are given in the Report of a number of full-scale tests on th fire-resistance of structural elements, various materials and protecti casings being employed.

The programme of tests on highway bridges, undertaken for th Ministry of Transport, has been completed during the year by the testi to destruction of Alcester Road South bridge, Birmingham, and by

eries of tests made on seventeen bridges with a heavily-loaded vehicle. Laboratory tests have also been made on specimens of materials removed from the bridges previously tested to destruction, and the results are now being analysed. The research on voussoir arches undertaken at the Imperial College of Science and Technology has been continued by tests on an arch of 10 feet span with concrete voussoirs; tests were made with lime-mortar joints, cement-mortar joints, and dry joints. The general results show that the arch has a considerable reserve of strength after the appearance of the first crack.

Progress during 1938 in the investigation into the impact pressures exerted by breaking waves on vertical sea-walls, which is being carried out by Dr. C. M. White and Major R. A. Bagnold at the Imperial College of Science and Technology, is described in the Report *.

In connexion with wind pressure on buildings, a series of tests has been carried out on a composite model representing a ½-mile square of closely built-up area to determine the effects of interference due to adjacent buildings. The work of analysis is not yet finished, but it appears that the wind-loads on a complete structure at the centre of such an area are probably not more than half of what they would be in a fully-exposed site.

The Report gives particulars of several investigations in progress on problems of soil-mechanics; in particular, considerable success is being attained in the analysis of the stability of earth slopes. The methods employed for analysing the settlement of structures are being checked by careful investigation of the foundation-conditions of two important new bridges; the settlements measured during and after construction will provide a valuable comparison with the calculations. The work on soil-mechanics is being supported by the main-line railway companies, the London Passenger Transport Board, and The Institution.

A number of investigations in hand cover various aspects of the efficiency of buildings from the standpoint of the user. In particular, questions of thermal and accoustic insulation, ventilation, condensation, and illumination are being studied. In addition, a large number of special investigations have been carried out for various trade organizations, commercial firms, consultants, Local Authorities, and Government Departments.

REPORT OF THE WATER POLLUTION RESEARCH BOARD FOR THE YEAR ENDED 30 JUNE, 1938 †.

The Annual Report of the Walter Pollution Research Board again emphasizes the necessity for constant vigilance on the part of those responsible for the supply of water to the public. Particular mention is made of the lessons to be learnt from the serious outbreak of typhoid

* An Interim Report on this work drawn up by Major R. A. Bagnold is printed on pp. 202–226, *ante*.

† Published by H.M. Stationery Office, price 1*s*. 0*d*.

fever at Croydon. Researches undertaken under the supervision of the Board include investigations on the treatment of water for public supply and for industrial purposes, on the treatment and disposal of domestic sewage and industrial effluents, and on problems of pollution of rivers. Some of them are briefly discussed in the following notes.

The discovery in the course of the Board's work that acids, bases, and salts can be removed from solution in water by means of certain synthetic resins has aroused widespread interest, and considerable progress has been made in the development of the process for industrial purposes. The main applications of the resins up to the present have included the treatment of water to remove all or part of the dissolved salts ; this had previously been possible only by more expensive methods, such as distillation. Utilization of the resins for the removal of valuable substances such as metals from very dilute solutions, for example from industrial effluents, and for the removal of objectionable substances in very low concentration in water is also being developed rapidly. Recent work under the Board has shown that, under certain conditions, fluorides which are sometimes present in water in small concentrations may be removed by resins of one type ; the fluorine is not readily removed, however, when other salts usually found in natural waters are present.

The investigation to determine the average quantity of lead taken up by certain types of water from lead pipes and fittings under conditions of household supply has been continued, and tests have been made on seventeen services in different parts of England and Scotland. Experiments are now in progress on methods of treatment of waters with the object of reducing their action on mains and service pipes of different materials.

One of the most important of the Board's investigations is the work which is being carried out, in collaboration with the milk industry, on the treatment and disposal of waste waters from dairies and milk-product factories. In 1935, after preliminary laboratory experiments, the Board erected two large-scale experimental plants at a milk-collecting and distributing depot and cheese factory at Ellesmere, Shropshire. Previous Annual Reports have shown that milk washings from the depot, which are difficult to treat by the method of biological filtration as ordinarily applied at sewage-disposal works, could be satisfactorily purified when passed at a controlled rate through two percolating filters in series, the order of the filters in series being reversed periodically in order to prevent the accumulation of excessive amounts of solid matter in the filters. During the past year the waste waters treated have contained whey washings, which are produced during the manufacture of cheese ; these washings are rather more difficult to purify than milk washings, but effluents of excellent quality have been obtained. The new method of operating percolating filters is now in satisfactory operation at a number of milk depots and cheese and butter factories. It has also aroused great interest among those concerned with the treatment and disposal of town sewage ; the Birmingham Tame and Rea District Drainage Board has placed a new laboratory and

itable large-scale plant at the disposal of the Board in order that treat-ent of sewage by the new method may be tried on a large scale.

The investigation on the discharge of sewage into the estuary of the ver Mersey, which was begun in 1933 and occupied about 4 years, has en completed, and a comprehensive special report has been published as ater Pollution Research Technical Paper No. 7. A full summary of the periments, observations, and conclusions is included in the present eport of the Board. The work was undertaken at the request of various cal interests, who met the whole of the cost (about £26,000); it has led the conclusion that the crude sewage discharged into the Mersey has no preciable effect on the amount and hardness of the deposits in the uary.

Other investigations described in the Report are concerned with the rious processes in methods of sewage-treatment and with the bacteriology water-supplies.

IE RESEARCH WORK OF THE INSTITUTE OF WELDING: JUNE, 1939.

The Symposium on the Welding of Iron and Steel held in 1935, in ich seventeen technical societies and institutions participated, focussed ention on the need for co-ordinating research on welding and for tiating additional work to make available the advantages that might be ained by its more general use. The Symposium Committee finally ommended that research on welding should be carried out under the itrol of a standing Research Committee appointed by the Council of the titute of Welding, and representative, as far as possible, of other erested organizations, which should be responsible for obtaining the essary funds. The Research Committee was constituted early in 1937; ias made a close survey of the requirements of industry in respect of welding of iron and steel and the principal non-ferrous metals, and a gramme of research covering the application of welding over the whole d of industry has been started.

Most of the problems brought out by the Symposium have now been ined more or less closely. They may be divided into three general sses: problems connected with the weldability of the various ferrous non-ferrous metals of construction; problems connected with the nnique of the application of the various processes; and problems ing from the special characteristics of welded joints as they affect the hod of design of the structure, machine, or vessel.

The success of all applications of welding depends on the weldability of material on which the welds are made. In the case of mild steel it is unate that the properties of the material are so little affected by the lication of heat that it may be said to be weldable without precautions. st other metals and alloys, however, are subject to local alterations er the welding heat. An investigation is therefore being made on

eleven typical high-tensile steels to evaluate the hardening at the wel junction, and since the hardened zone at the weld is very small an attemp is being made to produce a similar type of structure artificially on a larg scale by means of taper quench specimens, in order to enable the properti of the hardened material to be investigated fully. A study is also bein made of the effects on weldability of variations in the grain-size of stee (due to different methods of production) and of variations in the weldin technique.

The weldability of selected alloy-steels used for pressure-equipment being specially studied, each steel being considered in the form in which is most widely used; metal arc welding, gas welding, and resistance fla welding are to be covered. The welding of pressure pipe-lines is receivin particular attention, the precautions necessary to avoid the production various defects, and the value of various methods of inspection, bein studied.

Defects in the welding of intermediate alloy steels are often traceab to some inherent metallurgical feature; for example, certain chromiu steels may suffer from the formation of numerous fine cracks close to th weld, owing, it is believed, to the phase-changes and consequent volum changes. Experimental work on these problems is being carried out Birmingham University, and includes the mechanical and microscop examination of welds and the investigation of dilatation and hardenabilit

The welding of cast iron has not the same commercial importance the welding of steels, but much repair work is done on this material; t principal difficulties are therefore being studied in the laboratories of t British Cast Iron Research Association, specimens being prepared in t foundries of firms which are collaborating.

The welding of non-ferrous metals is of particular importance in vie of their use in many cases where a high resistance to corrosion is desire in such cases, the use of welding avoids the introduction of dissimil metals, as, for instance, by soldering. A study of existing informati has been carried out at the laboratories of the British Non-Ferrous Met Research Association, and experimental work is being commenced with investigation into the effect of small quantities of impurities on the wel ability of aluminium and copper.

The phenomena of weld shrinkage and internal stress are of maj importance in most welding operations, and fundamental research on the problems is being carried out at King's College, Newcastle. An attem is being made to establish a standard test for evaluating the liability weld-metal to crack during deposition, and consideration is now bei given to a machine which rotates one plate of a fillet-weld test piece duri the making of the weld so as to subject the cooling metal to a kno strain. Preliminary tests indicate that this method may afford a relia indication of the liability of different weld-metal deposits to cracking.

Experimental work on arc-welding procedure is directed towards t establishment of standard rules by means of which reliable welding may

sured. Faults in welds may be divided into two principal classes: the sible faults, such as incorrect profile, undercutting, overlapping, and surce holes; and the hidden faults such as lack of root penetration, lack of sion, porosity, cracks, and slag-inclusions. The visible faults present difficulty in detection, and it is considered that proper control of produre will guard against the occurrence of hidden faults. The First terim Report on this research has been published [1] and indicates that equate root penetration may be ensured by the use of a gauge of electrode operly proportioned in relation to the size of the fillet applied.

Methods of preparation of all-weld-metal test specimens are being idied to determine how truly representative specimens may be proced under industrial conditions.

The application of welding to shipbuilding has made steady progress ring the last 8 years, and its advantages are generally recognized, but perimental work is required in connexion with the rules governing its e. An investigation has accordingly been put in hand to compare the operties of welded and riveted structures in ships, to produce design ta for welded construction, and to ascertain the soundest, lightest, and ost economical methods of welded construction which can safely be subtuted for existing riveted types of bulkhead stiffeners, deck beams, shell iming, etc. The investigation is being supported by the Shipbuilding nference, the Institution of Naval Architects, and the Institution of igineers and Shipbuilders in Scotland. Special testing equipment allows ids up to 180 tons to be applied to specimens up to 24 feet long and 9 feet de; this equipment has been fully described [2]. An extensive investition has already been carried out on comparable sections of bulkheads th welded and riveted construction, and the deflexions of stiffeners, etc., deep tank bulkheads of a ship under construction have been observed der water-pressure conditions.

In framed structures one of the principal advantages that can be gained the use of welding is the increased rigidity obtainable in connexions. le Steel Structures Research Committee investigated the behaviour of jidly jointed steel frames within the elastic range, and this work is now ing extended by the Institute of Welding into the plastic range, in which eld occurs at certain sections. Small-scale frames are being tested to struction at Bristol University, and it is hoped that it will be possible to rmulate a general theory which can be checked later by comparatively v tests on full-scale structures. A First Interim Report, including an count of the first series of tests on model beams and model welded portalimes, has already been published [3].

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[1] First Interim Report of the Panel on Arc Welding Procedure. Trans. Institute Welding, vol. 2 (1939), No. 2.

[2] First Interim Report of the Panel on Ship Structures. Trans. Institute of ilding, vol. 2 (1939), No. 1, pp. 40–44.

[3] First Interim Report of the Panel on Framed Structures. Trans. Institute of ilding, vol. 1 (1938), No. 4, pp. 206–226.

Various problems are being studied in connexion with the use of weldin in plate-girder bridges; in particular, the stability of compression flang is being studied experimentally at the Engineering Laboratory of Cam bridge University. Although a considerable amount of theoretical wor has been carried out on the stiffening of plate-girder webs, the amount experimental work is small and it has proved difficult to apply the informa tion gained to actual design. An experimental investigation on th stability of webs in welded plate-girders is accordingly being made at th Imperial College of Science and Technology.

Considerable economies may be anticipated from the efficient use welding in structural details, and attention is therefore being given t beam-to-column connexions, column sections, economical column base beam-to-beam connexions, and details at joints of rigid frames. Exper mental work on these problems is being carried out at the Engineerin Laboratories of the University of Birmingham.

Many aspects of resistance welding are receiving attention. In pa ticular, the spot-welding of structural-steel material of the types used i building construction is being investigated at the instance of the Londo County Council. The spot-welding of light alloys is of growing importan in aircraft construction; spot-welds in "Alclad" and "Duralumin have already been tested, and the work, which is being carried out at th laboratories of the Royal Aircraft Establishment and the Norther Aluminium Company, Ltd., is being extended to other light alloys. Spo welded construction in 18/8 stainless steel is also being studied. For proje tion-welding, attention is being concentrated on the standardization of sui able projections for sheet-thicknesses ranging from 0·02 inch to 0·375 inch

Committees are now being set up to consider problems arising from th use of welding in pressure-vessels, and to study the fatigue of welde connexions.

In addition to the work of the various Committees, the Institute Welding maintains an up-to-date information service, in collaboration wit the intelligence services of co-operating bodies. The results of the researc work carried out are made available through the Transactions of the Inst tute of Welding, and also in the form of research reports publishe separately. A quarterly "Welding Literature Review" gives abstrac of the most useful published articles.

The Institution has made regular financial contributions to the researc work of the Institute of Welding, and members of the Research Committe and other members of The Institution are assisting in the work of the variou Committees.

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